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FIG. 1. VIEW OF TUOLUMNE RIVER, SHOWING LA GRANGE DAM AND CANALS.

(Modesto canal and wasteway at left; Turlock canal and wasteway in right foreground; La Grange Water & Power Co.'s canal and wasteway in right background.)

Discharge Measurements aud Formulas for Some Large Overflow Dams.*

By W. F. MARTIN,† Assoc. M. Am. Soc. C. E.

Because of the very few experimental data bearing on the subject of discharge over actual dams remarkably little is definitely known concerning the coefficients that should be applied to different types of overflow dams used as weirs. Yet it is probably well within the truth to say that Francis' formula for sharp-crested weirs, or some other sharp-crested weir formula, is well-nigh universally used in computing the discharge over various types of dams. This is true despite the fact that many experiments indicate that any particular formula has but a limited range and is not applicable to conditions other than those under which it was derived. The following data bear directly on the subject, and they are presented with the hope that they will prove interesting and useful to hydraulic engineers.

CALIFORNIA STREAM GAGINGS.—From June, 1906, to November, 1909, the writer was identified with the stream-gaging work of the U. S. Geological Survey in California, and in the regular course of his work made numerous gagings of practically all the streams of the State, thus acquiring much first-hand information concerning their general characteristics and regimen of flow.

The principal California streams rise in the Sierra Nevada, in the eastern part of the State, and flow westward into the Great Valley where they join the Sacramento or the San Joaquin river system. These streams have steep slopes throughout the greater part of their courses. They have also a regular late-spring and early-summer rise, resulting from melting snow. On account of their great fall and the quantity and seasonal distribution of their flow, they are of great importance commercially, chiefly for power and irrigation.

In the foothills permanent gaging stations have been established on all these streams by the U. S. Geological Survey. Every year discharge measurements are made on each stream in sufficient number to determine the station rating curve. Because most of the streams have channels that shift more or less, gagings must necessarily be continued from year to year to insure good results. It becomes very desirable, therefore, to utilize existing permanent dams for measuring the discharge, if they are of such a type as to permit of being rated. Particularly are such structures useful in determining the flow during floods and winter freshets, which are often very severe and of short duration.

On two streams of the San Joaquin system, permanent overflow dams of concrete-masonry exist near the gaging stations. One of these dams is in the Tuolumne River about 1½ miles above the town of La Grange, opposite which is the current-meter gaging station. It is shown by two views, Figs. 1 and 2. It is a high diversion dam, spanning the river between almost vertical rock walls extending far above its crest. It is curved upstream in plan and has an ogee section. The other dam, Figs. 3 and 4, is in the Merced River at Merced Falls about 1½ miles below the gaging station. It is a low dam stretching across a wide section of the river at the head of the rapids. This dam is straight in plan and has a broad flat crest. These two dams are good examples of distinct types working under different conditions. Their cross-sections are given in Fig. 5, together with those of two other dams for which discharge measurements exist.

In order to use these dams for determining the flood flow of Tuolumne and Merced rivers and in order to throw some light, if possible, on the proper coefficient of discharge to be used for such types, a special effort has been made to rate them by means of the regular gaging stations in the rivers. During the last four years, therefore, whenever current meter gagings of each river were made, the head on the dam was read simultaneously, and the measured discharge, corrected for diversion and leakage, was used as the flow over the dam under the observed head. In this way 25 measurements of the flow over the La Grange dam have been made, with heads ranging as high as 7 ft. and discharges as great as 17,000 cu. ft. per sec. Likewise 35 measurements of the flow over the Merced Dam have been made, with heads ranging as high as nearly 4 ft., and discharges as great as 11,400 sec. ft. The type of dam, the conditions existing at each, and the method of determining a rating equation from the measurements, are considered in the following paragraphs.

LA GRANGE DAM.—This dam has a height of 129 ft. above the lowest point of bed rock. It has a top width of 11 ft. and a bottom width of approximately 90 ft. The upstream edge of the crest is curved to a radius of 3 ft., and the downstream edge to a radius of 17 ft., the two curved surfaces being joined by a practically horizontal crest several feet in width. In plan the dam is curved upstream to a radius of 300 ft., and it has a length of 336 ft. at the top between the rock walls. The crest profile was intended to be level, but nearly half its length is about 0.2 ft. higher than the rest. The overflow length is 293 ft. at the crest, normally. The length increases somewhat with high heads, due

*The data included in this article were collected by the Water Resources Branch of the U. S. Geological Survey in cooperation with the state of California, under the direction of W. B. Clapp, district engineer, and are published by permission of the Director of the Survey.

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to the fact that the right end of the dam terminates in the concrete bulkhead of the Modesto irrigation canal in a series of four steps, having an average rise of 4.4 ft., and a tread of 5 ft. each. The left end wall is rock, approximately vertical, but very rough and jagged.

The La Grange dam was built jointly by the Turlock and Modesto irrigation districts to divert water from Tuolumne River for irrigating land on both sides of the river many miles below. Construction was begun in 1891, and the dam was completed in December, 1893. It is a concrete structure, faced with 3 ft. of granite rock masonry, except on the top, which is solid concrete to a depth of 6 ft. The dam required 29,000 bbls. of cement for its completion and cost $550,000, half of which was paid by each of the irrigation districts.

Turlock canal diverts water through a short tunnel from the left bank of the river, the headgates being only a few feet above the dam. Modesto canal takes water from the right bank through a concrete bulkhead at the end of the dam. Both canals waste water back into the river through concrete gates a short distance below the headings. (See Fig. 1; the view also shows water wasting from the La Grange Water and Power Co.'s Canal, which heads about 15 miles above the dam.)

The Turlock Canal is owned by the Turlock Irrigation District and supplies water for the irrigation of land in the San Joaquin Valley on the south side of Tuolumne River. The district has 176,500 acres under irrigation, and has a bonded indebtedness of $1,400,000. Its water appropriation amounts to 1,500 sec. ft., but not more than two-thirds of that amount has yet been diverted.

The Modesto Canal belongs to the Modesto Irrigation District and supplies water for irrigating land in the San Joaquin Valley on the north side of Tuolumne River. The Modesto District has 81,200 acres under irrigation, and a bonded indebtedness amounting to $1,350,000. Its appropriation amounts to 640 sec. ft., nearly all of which has already been diverted. During the fall of each year the total flow of the river is notably less than the combined capacity of the canals, and the crest of the dam is dry for a considerable period.

MERCED FALLS DAM.—The Merced Dam, Figs. 3 and 4, is a low, broad-crested concrete structure. It is straight in plan, has a vertical upstream and a plane sloping downstream face, and a flat crest with practically sharp corners or edges (see Section in Fig. 5). It has an overflow length of 464 ft. at the crest, a top width of 6 ft., and a height of approximately 23 ft. (The exact height could not be ascertained.) Each end of the dam terminates in a concrete bulkhead which rises through a series of steps to a height of 8 ft., above the crest. The bulkhead at the left end makes an angle with the dam of about 45° upstream, but the bulkhead at the other end is straight. The crest profile was undoubtedly designed to be level, but the central portion is about 0.2 ft. higher than the rest (see profile in Fig. 6; the shallow notch in the center is opposite the fish ladder on the lower side). The notches along the down-stream edge, seen in Figs. 3 and 4, are nearly 2 ft. square and about 2 ft. deep, and were designed originally for use in connection with a system of automatic hydraulic flashboards which have never been installed.

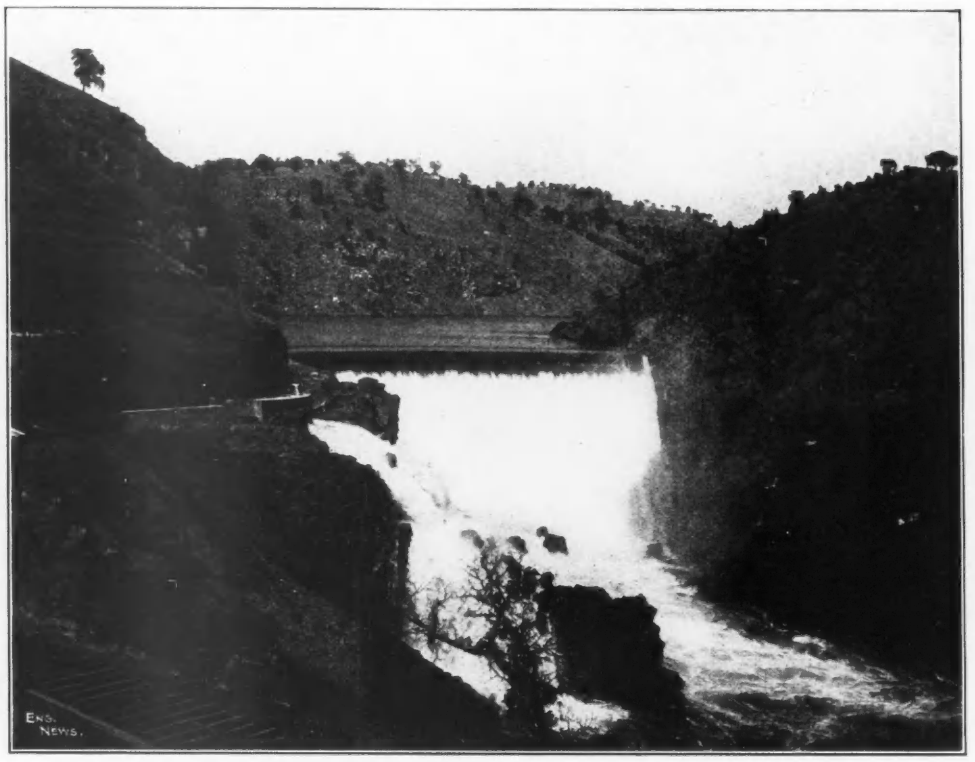

FIG. 2. NEAR VIEW OF LA GRANGE DAM ON TUOLUMNE RIVER.
(Modesto canal at left.)

This dam was constructed a few years ago for power development by the Merced Falls Gas and Electric Co. The power house is located on the right bank just below the dam, and the water is delivered to the wheels through a 20-ft. millrace at the end of the bulkhead. The total diversion around and through the dam never exceeds 200 sec. ft., and consists of the amount used for power and small quantities used for irrigation on each side of the river. The crest of the dam is usually dry for a considerable period during the fall of the year.

Fig. 4. At Normal Stage. Fish Ladder in Middle of Dam.

Fig. 3. At Moderately High Stage of River.
FIGS. 3 AND 4. DAM AT MERCED FALLS, MERCED RIVER.

Rating Curves and Equations.

Every gaging furnished two figures: the head on the dam in feet, and the discharge in cubic feet per second. As each measurement gave different values of the two quantities, it is an easy matter to construct a satisfactory rating table covering the range of observed values by simply plotting the discharges as abscissas and the heads as ordinates, and drawing in the mean curve, as has been done in Fig. 6. Such a curve, however, is subject to great uncertainty when extended very much beyond the range of observation, since the rate of curvature changes at every point. Also, from such

TABLE I.—MEASUREMENTS OF FLOW OVER THE LA GRANGE DAM.
Current-Meter Measurements Made at Bridge 1½ Miles Below the Dam.

No.	Date.	Hydrographer.	Head on dam in ft. H.	Disch. in sec. ft. Q.	Coefficient c*	Per cent. error§
	1906.					
1	Feb. 18	C. H. Lee	1.64	1,860	3.020	— 2.0
2	Mar. 29	R. S. Hawley	3.76	6,210	2.907	— 5.7
3	Apr. 3	R. S. Hawley	3.54	5,700	2.921	— 5.4
4	May 15	C. H. Lee	5.02	11,000	3.338	+ 8.1
5	26	W. C. Sawyer	4.32	7,740	2.942	— 4.9
6	28	"	6.96	15,900	2.956	— 4.5
7	June 4	"	5.37	11,100	3.045	— 1.6
8	14	"	6.54	14,800	3.020	— 2.3
9	21	"	7.06	17,000	3.093	— 0.1
10	29	"	4.81	9,720	3.146	+ 1.9
11	July 3	"	7.33	18,300	3.148	+ 1.7
12	11	"	5.24	11,800	3.356	+ 8.6
13	21	"	4.36	8,540	3.202	+ 3.5
14	31	"	2.75	4,120	3.116	+ 0.8
15	Aug. 3	"	2.06	2,670	3.082	— 0.2
16	6	Sawyer & Martin	1.62	1,860	3.078	— 0.2
17	20	W. F. Martin	1.26	1,350	3.256	+ 5.8
18	Nov. 21	"	.30	177	3.077	+18.8
	1907.					
19	May 28	W. F. Martin	5.06	10,900	3.268	+ 5.9
20	31	"	6.18	14,400	3.199	+ 3.6
21	Aug. 1	Wm. A. Lamb	2.31	3,040	2.955	— 4.4
22	16	W. F. Martin	1.08†	913	2.777	— 9.9
	1908.					
23	Mar. 3	W. F. Martin	1.18†	1,000	2.661	—13.2
24	May 1	Wm. A. Lamb	3.30	5,370	3.057	— 0.9
	1909.					
25	June 2	W. F. Martin	6.10	13,600	3.083	— 0.4

*In formula $Q = c\,l\,h^{3}/_{2}$.
†These gage heights are undoubtedly too high.
§Referred to mean logarithmic rating.

TABLE II.—MEASUREMENTS OF FLOW OVER THE MERCED DAM.
Current-Meter Measurements Made at U. S. Geological Survey Cable Gaging Station About 1½ Miles Above Dam Except as Noted.

No.	Date.	Hydrographer.	Head on dam in ft. H.	Disch. in sec. ft. Q.	Coefficient c*	Per cent. error§
	1906.					
1	June 22	W. C. Sawyer	3.95	10,800	2.964	—17.9
2	July 13	"	2.90	6,730	2.940	—11.6
3	20	"	2.00	3,970	3.026	+ 0.8
4	21	"	1.90	3,820	3.144	+ 6.1
5	Aug. 19	W. F. Martin	.75	737	2.446	+ 5.7
6	Sept. 6	"	.32	117	1.392	—24.5
	1907.					
7	Apr. 30	"	2.60	6,120	3.149	— 2.4
8	May 27	"	2.70	7,420	3.606	+10.7
9	31	"	3.35	10,100	3.550	+ 2.7
10	June 1	"	3.62	11,400	3.569	+ 1.2
11	July 6	"	2.40	5,200	3.015	— 4.4
12	8	W. V. Hardy	2.38	5,380	3.158	+ 0.3
13	9	"	2.24	4,770	3.065	— 1.1
14	10	"	2.03	4,160	3.099	+ 2.8
15	11	"	1.96	3,600	2.829	— 5.4
16	15	"	1.85	3,380	2.895	— 1.6
17	17	"	1.68	2,910	2.880	+ 0.3
18	20	"	1.55	2,390	2.668	— 5.0
19	22	"	1.40	2,170	2.824	+ 3.5
20	23	"	1.34	1,810	2.632	— 1.6
21	29	Wm. A. Lamb	1.30	1,900	2.762	+ 3.3
22	30	"	1.18	1,580	2.656	+ 1.9
23	30	"	1.15	1,440†	2.518	— 2.9
24	31	"	1.10	1,430	2.673	+ 4.5
25	Aug. 3	"	1.02	1,200	2.512	+ 0.2
26	5	"	.96	1,110	2.544	+ 3.1
27	6	"	.90	964†	2.434	+ 0.5
28	10	"	.70	594†	2.188	— 3.4
29	11	"	.74	666	2.252	— 2.2
30	15	W. F. Martin	.67	565†	2.220	— 1.1
	1908.					
31	Mar. 2	W. F. Martin	.99	1,100	2.408	— 3.2
32	May 2	Wm. A. Lamb	1.85	3,400	2.912	— 1.0
33	June 17	W. F. Martin	.99	1,130	2.472	— 0.5
	1909.					
34	May 17	W. F. Martin	2.15	4,350	2.973	— 3.0
35	June 3	"	3.72	10,300‡	3.094	—12.9

*In formula $Q = c\,l\,h^{3}/_{2}$.
†Measurements made on the crest of the dam.
‡Measurement carefully made. There was some obstruction on the crest which increased the gage somewhat.
§Referred to mean logarithmic rating.

a curve it is difficult to determine the properties of the equation of rating sufficiently well to exhibit the law of variation of discharge with change of head.

From what has been said above, it is evident that some other method must be used in order to obtain reasonably accurate deductions for high heads. Indeed a form of curve is needed that can be extended with reasonable certainty to any desired value beyond the range of observation. This requirement appears to be well satisfied by the logarithmic transformation of the simple exponential equation

$$Q = mh^n$$

where

Q = discharge in second-feet,
h = head on dam in feet,

and m and n are constants to be determined irrespective of the velocity of approach, or length, shape, or condition of crest.

By writing the logarithm of each side, the exponential equation becomes,

$$\log Q = n \log h + \log m.$$

If m and n are constants, this logarithmic equation is precisely analogous to the simple linear equation $y = ax + b$, and will appear as a straight line if the logarithms of the quantities are plotted instead of the quantities themselves. The actual slope of the line with reference to the axis of log h will determine the value of n, and the intercept on the axis of log Q, for $h = 1$, will give the logarithm of m, from which m can be taken from a table.

The results of the measurements of flow over the La Grange and Merced dams, are given in Tables I and II. These also show in a separate column the coefficients derived from the theoretical formula $Q = clh^{3/2}$ and in the next column the percentage error of each measurement referred to the mean logarithmic rating.

The measurements are plotted to logarithmic coordinates in Fig. 7. It is easily seen that the plotted points for each dam are well satisfied by a mean straight line whose equation is

$$Q = 905h^{1.50} = 3.088\,lh^{1.50}$$

for the La Grange dam, and

$$Q = 1156h^{1.77} = 2.491\,lh^{1.77}$$

for the Merced dam.

Each line is well defined between certain limits, which are fixed by the conditions governing accuracy at the gaging stations. The most favorable conditions for current-meter gagings on Tuolumne River exist when the head on the dam ranges between 1.5 and 7.0 ft. This range, however, does not include more than one-third of the known range of the river, which has exceeded 16 ft. But it does include practically all the trustworthy measurements from which a rating for the entire range must be made. The logarithmic straight line is fixed by 21 such measurements, and has, therefore, the greatest possible weight. The only doubt that can arise concerning its accuracy at extremely high stages involves its linearity.

On Merced River the conditions most favorable for meter measurements do not embrace such a wide range of stage on the dam, nor, indeed, is the dam known to have had more than 7 ft. over its crest. The logarithmic straight line is determined by 28 measurements made at heads ranging from 0.5 to 2.6 ft., some of them being made on the crest. Several measurements at higher stages are given, but they are more or less uncertain, owing to the high velocity at the gaging station. It is evident that this line is not so well determined as that for the La Grange dam, and that it can not be extended so confidently. Indeed, it may be that the exponent is somewhat too high, but more data are needed before this can be known with certainty.

Considering the fact that the measurements were made by several different persons with different instruments and under different conditions of rising or falling stage (the Sierra streams at intermediate stages are seldom stationary), one can hardly expect them to be more consistent among themselves than they are. It is well known that Price meters, such as were used for these measurements, will check the best sharp-crested weirs within 3 to 5% under good conditions. Under the prevailing conditions in these rivers it appears, therefore, that the measurements are reasonably accordant and furnish reliable ratings for the dams as broad-crested weirs. Certainly the ratings are more satisfactory for computing flood flow than the ordinary station rating curves.

A rating table computed from the above determined equation and covering the entire known range of the stream is given in Water Supply Paper No. 251 of the U. S. Geological Survey for the La Grange dam. It is hoped that such tables will serve permanently in the future to determine the flow in Tuolumne and Merced rivers, it being necessary to make only occasional meter measurements for comparative purposes. For heads of less than one-half foot the tables are subject to more or less error, and are, of course, unreliable.

A comparison at different heads of the flow per linear foot and of the value of c computed on the basis of the 1.5 power of the head is presented by Table III. This reveals some interesting differences. Certain differences, of course, would very naturally be anticipated from a knowledge of the dissimilar types of dam, but it is hardly possible that the extent and mode of variation would be foreseen. At low heads the disproportionately large difference probably results from a slight error in the datum of Merced gage; and at high heads the difference is due primarily to the effect of upstream curvature in La Grange dam, which has the effect of causing the filaments to converge in passing over the crest, forming noticeable pressure ridges on the surface.

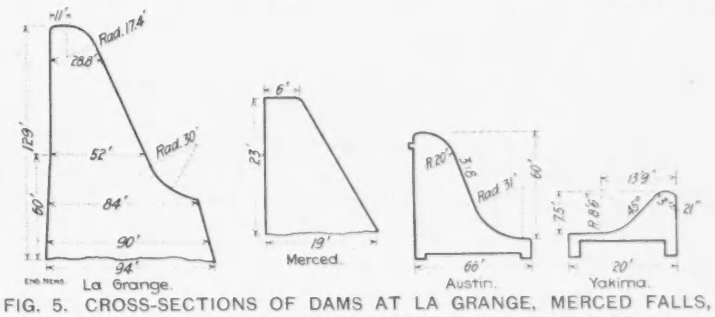

FIG. 5. CROSS-SECTIONS OF DAMS AT LA GRANGE, MERCED FALLS, AUSTIN (TEX.) AND YAKIMA. [NOT TO SAME SCALE.]

TABLE III.—COMPARISON OF THE VALUE OF c IN FORMULA $Q = c\,l\,h^{3}/_{2}$, AND OF THE DISCHARGE PER LINEAL FOOT, FOR MERCED, LA GRANGE, AUSTIN AND YAKIMA DAMS.

Head,	Merced Dam		La Grange Dam		Austin Dam		Yakima Dam	
ft.	c.	Q_1	c.	Q_1	c.	Q_1	c.	Q_1
0.5	2.065	.730	3.088	1.091	3.003	1.062	2.803	.991
1.0	2.491	2.491	3.088	3.088	3.196	3.196	3.110	3.110
1.5	2.780	5.108	3.088	5.066	3.315	6.090	3.305	6.072
2.0	3.003	8.49	3.088	8.74	3.402	9.62	3.451	9.76
2.5	3.190	12.61	3.088	12.22	3.473	13.72	3.568	14.10
3.0	3.352	17.41	3.088	16.04	3.524	18.31	3.668	19.06
4.0	3.623	28.99	3.088	24.71	3.620	28.96	3.829	30.64
5.0	3.849	43.02	3.088	34.50	3.694	41.30	3.960	44.27
6.0	4.041	59.40	3.088	45.40	3.754	55.19	4.071	59.82

The velocity of approach in each case is undoubtedly high, on account of the shallow reservoirs back of the dams. In the fall of 1909 the writer visited the La Grange dam, when the water surface was about 2 ft. below the crest. On that occasion a bank of sand and silt rose well above the water over a large part of the former small pond, and it was said by reliable persons that a man could wade almost anywhere in the remaining channel except for a few feet just back of the dam, where the water was 8 or 10 ft. deep. Velocity of approach under such circumstances would necessarily be high. It is probable also that the water back of Merced dam averages only a few feet in depth below the crest, and the high velocity of approach resulting therefrom accounts for the rapid increase in the value of *c*.

A comparative study of the great difference in the coefficients of La Grange and Merced dams has prompted the writer to seek for other similar instances of measurements of flow over actual dams. Two such instances have come to his notice, and the results are tabulated hereafter and analyzed precisely as for the La Grange and Mercer dams.

AUSTIN DAM.—The dam across the Colorado River at Austin, Texas, was a long high dam of ogee section. (See Fig. 5.) It had an overflow length of 1,091 ft. and a height of 60 ft. above low water. The crest profile was a compound curve formed by two circular arcs of radii 8 and 20 ft., respectively, the curve of short radius being upstream.

A series of current-meter measurements of the flow over this dam was begun in January and March of 1900 by Mr. E. E. Howard, a senior student in civil engineering at the University of Texas, under the direction of Prof. T. U. Taylor.* It was planned to continue the series until the entire range of the river was covered, but the failure of the dam in April, 1900, made this impossible. The gagings actually made cover only a small range of head, but several measurements were made at each of five different stages, which serve to fix the coefficients closely. The measurements and resulting coefficients are tabulated in Table IV., and the discharge curve in logarithmic scales is given in Fig. 7 with those for the other dams. The plotted points determine a well-defined straight line whose equation is

$$Q = 3487\ h^{1.59} = 3.196\ lh^{1.59}.$$

YAKIMA DAM.—The diversion dam for the Sunnyside Irrigation Project across Yakima River near Wapato, Wash., is a long low dam of ogee section. (See Fig. 5.) It has an overflow length of 500 ft. and a height of 7.5 ft. above the overflow apron, and its crest profile is a smooth curve of short radius. Near one end of the dam is a sluiceway 6 ft. wide and 7 ft. deep, which is closed by flashboards during low water.

During 1909 current meter measurements were made at a regular gaging station about 1,000 ft. below the dam, maintained by the U. S. Geological Survey in cooperation with the U. S. Reclamation Service.* At each gaging the head on the dam was observed by means of a hook gage set to the mean level of the crest, which had a variation of about 0.2 ft. between the highest and lowest points. The measurements covered a range of 6 ft. on the dam. They are given in the article quoted and rearranged here in Table V. The discharges in this table are for a length of 494 ft., the theoretic discharge through the 6 ft. sluiceway being deducted from the total discharge.

The logarithmic plot of the Yakima gagings is included in Fig. 7. It will be noted that the points scatter considerably at all stages, but particularly for those below one foot. This inconsistency is undoubtedly due in large measure to the disturbing effect of driftwood which lodges on the crest at all stages to a greater or less extent. The unduly large value of *c* for heads of less than half a foot probably results from error in allowing for the discharge through the sluiceway and possible leakage under or around the dam, and also from the fact that the gage datum is probably too high. In measurements Nos. 1, 2, 3 and 5, for which the values of *c* are excessively high, the sluiceway was closed by flashboards, and it is probable that leakage was very great. Only the last 12 measurements, including heads above half a foot, are used to determine the mean straight-line rating. The equation of this line is

$$Q = 1536\ h^{1.65} = 3.110\ lh^{1.65}.$$

SUMMARY.—To summarize and compare: The foregoing equations have been derived upon the assumption that the discharge varies as a fixed power of the head (not necessarily the 1.5 power). Or, expressed as an equation, $Q = mh^{n}$, where *m* and *n* are to be determined irrespective of the length, form, or condition of the crest, or of the velocity of approach. By logarithmic plotting of the gagings these constants are easily determined. The equations for the four dams discussed above are found to be as follows:

Merced,	$Q = 2.491\ lh^{1.77}$
La Grange,	$Q = 3.088\ lh^{1.50}$
Yakima,	$Q = 3.110\ lh^{1.65}$
Austin,	$Q = 3.196\ lh^{1.59}$

The real difference in these equations is best seen in Table III, where the discharge per linear foot and the coefficient *c* on the basis of the three-halves power of the head as generally used are given for different values of *h*. It is seen that, with the exception of La Grange dam, which stands in a class by itself on account of its curved plan, *c* increases very perceptibly with increase of head, and that this variation in *c* is greatest in the case of low dams, such as Merced and Yakima.

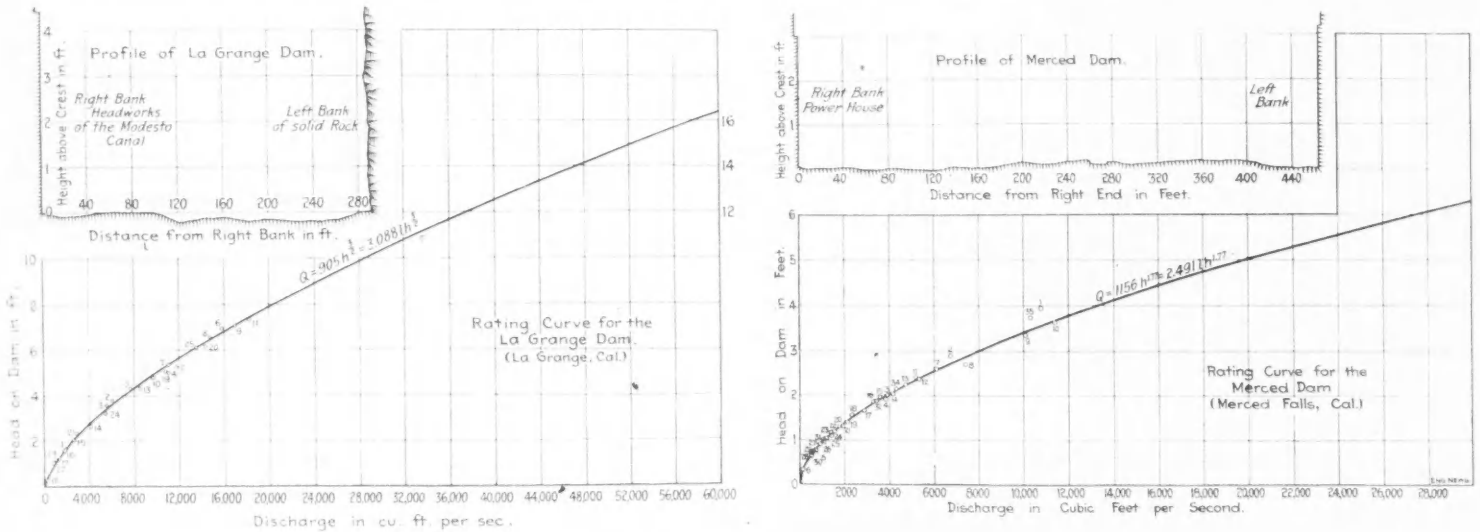
Profile of La Grange Dam.
Right Bank Headworks of the Modesto Canal
Left Bank of solid Rock
Height above Crest in ft.
Distance from Right Bank in ft.
Q = 3.40 h^1.59 = 2.088 l h^1.5
Rating Curve for the La Grange Dam. (La Grange, Cal.)
Head on Dam in ft.
Discharge in cu. ft. per sec.
Profile of Merced Dam.
Right Bank Power House
Left Bank
Distance from Right End in Feet.
Q = 1.156 h^1.77 = 2.491 l h^1.77
Rating Curve for the Merced Dam (Merced Falls, Cal.)
Head on Dam in Feet.
Discharge in Cubic Feet per Second.

FIG. 6. DISCHARGE CURVES FOR LA GRANGE AND MERCED DAMS.
Merced Dam. Straight in plan. Height 23 ft. Section trapezoidal. Crest width 6 ft.
La Grange Dam: Dam curved in plan, 300 ft. radius. Section ogee. Height 129 ft.

TABLE IV.—MEASUREMENTS OF FLOW OVER THE AUSTIN DAM.

Head on dam in ft. H.	No. of meas. at this stage.	Mean discharge in sec.-ft. Q.	Coefficient c*	Per cent. error†
.42	4	924	3.112	+3.9
.72	11	2,030	3.053	—2.4
1.09	4	3,890	3.132	—2.3
1.32	5	5,470	3.302	—0.9
1.44	3	6,280	3.333	—0.5

*In formula $Q = c\,l\,h^{3}/_{2}$.
†Referred to mean logarithmic rating.

TABLE V.—MEASUREMENTS OF FLOW OVER DIVERSION DAM, YAKIMA RIVER, NEAR WAPATO, WASH., SUNNYSIDE IRRIGATION PROJECT.

No.	Head on dam in ft. H.	Discharge in sec.-ft. Q.	Coefficient c*	Per cent. error†
1	.11	162	9.12	....
2	.11	189	10.60	....
3	.12	226	10.90	....
4	.14	104	4.04	....
5	.18	286	7.62	....
6	.30	479	5.91	....
7	.47	692	4.35	....
8	.88	1,190	2.92	—20.1
9	1.12	2,000	3.41	+ 7.5
10	1.23	2,500	3.71	+12.1
11	1.55	3,020	3.16	— 5.3
12	1.59	2,840	2.97	—14.4
13	1.85	4,680	3.78	+11.4
14	1.90	4,510	3.48	+ 2.8
15	2.57	7,940	3.90	+ 9.6
16	2.76	8,100	3.58	— 0.4
17	2.87	8,450	3.52	— 3.4
18	4.33	16,100	3.62	— 6.9
19	6.05	31,600	4.30	+ 5.7

*In formula $Q = c\,l\,h^{3}/_{2}$.
†Referred to mean logarithmic rating.

*See Water Supply and Irrigation Paper No. 40, p. 33.

*See article in Eng. News, Vol. 63, p. 481, April 28, 1910, by J. C. Stevens, Assoc. M. Am. Soc. C. E., Dist. Engr., U. S. Geological Survey, Portland, Oregon.

Referring again to the equations themselves, one is impelled to the conclusion that the discharge does not vary as the three-halves power of the head, but as some higher power such as 1.60 to 1.75. If it be true, however, that there is irresistible magic in the three-halves power, then it may be said that the coefficient c varies as some power of the head greater than zero, such as .10 to .25. In either case it is clear that Q increases more rapidly than $h^{3/2}$. This fact probably explains why some engineers insist that the current meter invariably gives too great a discharge when compared with broad-crested weirs. Possibly their formulas are in error.

In conclusion, it is interesting to note that the designers of the La Grange dam estimated the extreme maximum flow of Tuolumne River at 130,000 sec. ft. This flow would require a head of 27.4 ft. on the crest of the dam, which would be 10 ft. above the top of the head gates of Modesto Canal. The Austin dam at the moment of failure had a head of 11.1 ft. This meant a discharge of 160,000 sec. ft. At the estimated extreme flood stage of 16 ft. the discharge would have been 286,500 sec. ft., instead of 250,000, as estimated.

Quality of Lake Michigan as a Water-Supply for the North Shore District.

The increasing sewage pollution of Lake Michigan and the important relation of the sewage-disposal and water-supply problems to the health of the growing residential towns just north of Chicago, has led the North Shore Sanitary Association to have a sanitary survey made of this part of the lake. The association was organized about two years ago (Secretary, W. C. Hughes, Mayor of Glencoe, Ill.). It is a local auxiliary of the Lake Michigan Sanitary Association, which represents the states, municipalities, health boards, etc., interested in the use of the lake as a source of water-supply. The survey mentioned above was made by Mr. Langdon Pearse, Assoc. M. Am. Soc. C. E., Assistant Engineer of the Sanitary District of Chicago. Its results were presented in a paper read by him before the main association, and this has been printed in a bulletin of the North Shore Association, from which we take the following extracts:

The water in the north of Lake Michigan is hard, as the alkalinity is about 120 parts per million. This hardness gave us no indication of pollution, as it sometimes does. We did find, however, that the chemical results in general gave us but slight indication of the distribution of the pollution. The oxygen consumed, free ammonia, and albuminoid ammonia, although low in amount, were much higher inshore than out. While this may be due to wave action stirring up the bottom, it does point clearly to the dissemination of the polluting matter, and its consequent disappearance far from shore. This served, however, to verify the results which we obtained from our bacteriological analyses and convince us that the water-supply on the North Shore is distinctly polluted. The following are the results of our analyses:

(1) SHORE SAMPLES (400 to 500 ft. out).—In every case but one, out of 92, excessively high bacterial counts were often found. The counts ran from 384 to 30,000 per c. c., with one of 56 per c. c. B. coli were present more than half the time. The turbidity was high, in one case 180 parts per million. This means a muddy water.

(2) ONE MILE OFF-SHORE.—In 57 cases out of 73, the bacterial counts were high, ranging, however, from 10 to 35,000 per c. c. B. coli were present very often. The results from this sampling station are slightly better than from the inshore one, but in neither case is the water fit for drinking purposes, unless it be boiled, sterilized or filtered, on account of the presence of fecal pollution.

(3) TWO MILES OFF-SHORE.—In 35 cases out of 54 the bacterial counts are high, particularly near the foci of pollution, where B. coli are occasionally present. The entire range was from 15 to 35,000 per c. c. Near the smaller towns, the counts run from 30 to 6,500 per c. c. and B. coli appear seldom.

(4) THREE MILES OFF-SHORE.—In most cases the counts are fairly low, with B. coli seldom present, but around the foci of pollution the counts run higher, from 10 to 25,000 per c. c., with B. coli occurring occasionally.

(5) FOUR MILES OFF-SHORE.—The samples taken off Evanston show fair bacterial counts, from 30 to 6,140 per c. c.; Kenilworth and Winnetka, 12 to 1,400; Lake Forest, 25 and 400, with no B. coli in any of these cases. But around North Chicago and Waukegan the counts run from 100 to 16,500 per c. c., with one case of B. coli out of three samples at each place.

(6) FIVE MILES OFF-SHORE.—Samples were taken off Evanston and Waukegan, as these two cities discharge the principal amount of pollution. At Evanston the counts run from 30 to 760, with B. coli absent. Off Waukegan, the counts run from 70 to 10,000, with B. coli absent.

In a normal water to be used for drinking, bacilli should not run much over 300 per c. c., and B. coli ought always to be absent. The results we secured would indicate (*a*) that as far as fecal pollution goes, the water supplies on the North Shore are decidedly polluted, (*b*) that near a small town the intake should be located not less than two miles from shore, and (*c*) that around the larger towns the intake should be placed not less than four miles from shore. Such a conclusion, however, should be qualified, for in any case it is possible, with the right combination of winds, for fecal pollution to reach out even three and four miles, so that the only safety for a lake water supply is filtration.

Major Judson (First Report of the Lake Michigan Water Commission) concludes that there are no defined currents in Lake Michigan. My observations point to the same conclusion. The winds produce currents which may travel in any direction, and consequently must be reckoned with as carriers of pollution to any neighboring intake.

The results of the turbidity observations show that the turbidity varies with the depth, and that in all probability the bottom is stirred up by the action of the waves even at a depth of 40 ft. Beyond that depth, however, the action is slight, though in some cases we found there was a turbidity after a severe blow. There is, therefore, the additional reason of turbidity in studying the location of a water-works intake. In such a case, stress should be laid on the comparative importance of getting a clear water, because it is highly probable that a good filtered water can be made at less expense with an intake near shore, than by running the intake out sufficiently far to escape all turbidity and probable fecal pollution.

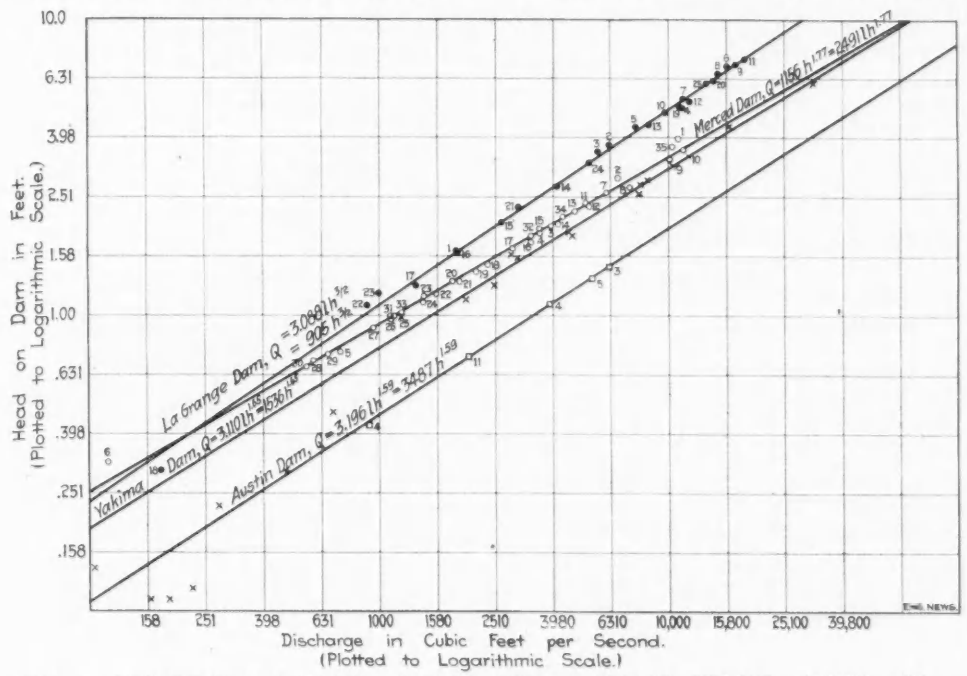
10.0
6.31
3.98
2.51
1.58
1.00
.631
.398
.251
.158
Head on Dam in Feet. (Plotted to Logarithmic Scale.)
La Grange Dam, Q = 3.089 l h^3/2
Yakima Dam, Q = 3.110 l h^1.85 = 1536 h
Austin Dam, Q = 3.196 l h^1.59 = 3487 h^1.59
Merced Dam, Q = 1156 h^1.77 = 24911 h^1.77
158 251 398 631 1000 1580 2510 3980 6310 10,000 15,800 25,100 39,800
Discharge in Cubic Feet per Second. (Plotted to Logarithmic Scale.)
ENG. NEWS.

FIG. 7. LOGARITHMIC DISCHARGE CURVES FOR LA GRANGE, MERCED, YAKIMA AND AUSTIN DAMS.

In inspecting the conditions on the North Shore, the most striking point is the recklessness with which sewage is discharged right at, or near, the water-works pumping stations. For instance, at Highland Park there is a septic tank with an 8-in. discharge right at the water-works, and the intake of the water-works is only about 2,000 ft. out in the lake. It is very probable that fecal pollution might reach the water-works intake, particularly as our results show that B. coli are present there. This particular tank is flanked by one about a mile north and another ¼-mile south, which also discharge right into the lake.

Doubtless the town officials thought that they were safeguarding the purity of the water-supply by building septic tanks. Unfortunately they did not, for the general effect of such tanks is to remove possibly 50% of the suspended matter, both mineral and organic. *The bacterial purification is frequently nil, as in many cases the number of bacteria leaving the tank is higher than the number entering it.*

At Evanston, the conditions will be changed when the new North Shore canal of the Sanitary District of Chicago is opened next fall, but at present, with the intake about 6,500 ft. out from the shore, the water at times is not safe to use.

In general, the sewage outfalls are so close to the intakes that it is only a question of a few years, with the rapid growth of population on the North Shore, before conditions will be distinctly worse, and the presence of B. coli will be noted all the time. My conclusion is, therefore, that these results show that under present conditions the water of Lake Michigan, from Evanston to Waukegan, is not a safe source of water supply at present. Nor is the physical character of the water as good as it should be, for during certain periods of the year the turbidity of the lake at the water-works intakes is excessive.

The remedy for this trouble is a question to be solved by the sanitary engineer, with a careful study of the costs in question. There are obviously several courses open. Either the sewage can be kept out of the lake altogether (through an extension of the Sanitary District canal or by a large intercepting sewer up the North Shore); or the sewage can be treated by sedimentation, sprinkling filters, and then sand filters, or possibly sterilization, discharging into the lake; or the water supplies can be filtered.

There is also the problem of the extension of the intakes, but as long as the sewage is allowed to be discharged freely into the lake, the extension of the intake is at best temporizing with the difficulty, for the pollution is always driving out in greater amounts and the turbidity will always be there. If the intake is run out so far as to escape both turbidity and pollution, I believe that the estimates will show that it would be cheaper to adopt filtration, to say nothing of the far greater safety thereby secured.

As to the policy of purifying the sewage before putting it into the lake, and then using the lake water, I believe that it will only be a few years before the lake water will have to be filtered, because the demand is growing for filtered lake water, clear and sparkling when it reaches the consumer. It is possible, with modern methods of sewage disposal, to make an effluent which is safe to drink, but it is best, all things considered, to keep sewage out of a drinking supply, for then the danger is removed of pollution through failure of an apparatus which is difficult to make fool-proof.

My conclusion, therefore, is, in a general way, that the solution of the problem of securing a safe water-supply for the North Shore towns is the filtration of the lake water, and the disposal of the sewage into the Sanitary Canal, either by a subsidiary canal or intercepting sewer. Filtration is easily possible, at moderate cost. In this case the evidence all points to the adoption of mechanical filtration, as the water is sufficiently hard and the sediment is, in many cases, so evenly and finely divided that a slow sand filter would probably not give an absolutely clear effluent at all times.

In connection with the matter of filtration, it is extremely important that a plant when built should be operated properly. This can best be secured by the concentration of the water-supply plant into one or more filter plants of large enough size to warrant the employment of an expert who can supervise constantly their operation. In that way only can perfect results be obtained and the protection secured which the North Shore residents desire.

As to the means for carrying out this work, there seem to be two ways available. One is for each town to do it individually. The other is that a water district be formed covering several towns. In this way the operating costs are lowered and the development and maintenance can be carried on under expert supervision,

whereas, with the individual development, no one town wants to go to the expense of the expert advice which is really necessary.

I would suggest that if the towns on the North Shore from Wilmette to Waukegan can get together and organize such a water board with full power to act; that they secure the best engineering advice available, and have worked out in every detail the various possible methods, for in that way only can a pure water-supply be secured at the lowest cost. Our results show that a pure water-supply is badly needed, for the present one is not only turbid, but at times is actually polluted. This conclusion is borne out by annual newspaper reports that typhoid fever is again setting in on the North Shore.

Cost of Filtration or Long Intakes.

The same subject has been discussed by Dr. E. Bartow, Director of the Illinois State Water Survey, and from his paper (given in the bulletin mentioned above) we take the following extract in regard to a comparison of the filtration and long-intake system.

The first step toward obtaining a pure water is to extend the intakes or to filter all of the water used. This should be done before the sewage is treated, if it should not be found possible to do both at once. Later it will be found advisable also to treat all sewage, so that the water filters will have less of the burden to bear. If the water intake could be extended far enough into the lake the danger from surface drainage would be eliminated and the danger from shipping would be small.

The extension of tunnels is expensive and this method can be used only when a large amount of water is demanded. The relative cost of filtration and of intake extension has been discussed by C. B. Burdick in a paper at the April meeting of the Lake Michigan Sanitary Association. Assuming that in any particular situation there is a certain distance from shore or pollution source where pure water may always be obtained, he states that in general a plant to supply 5,000,000 gals. per day can be obtained through a 24-in. main three miles off shore at approximately the same cost as filtered water can be obtained through a 16-in. intake extending ½-mile off shore. While the cost of the three-mile line is greater, the operating expense of the filter plant equalizes the annual charges.

In the case of larger quantities of water the situation is different. A 7-ft. tunnel, three miles in length, to supply 100,000,000 gals., can be constructed and operated at a cost of $24,000 per annum, while a filtration plant with short intake to supply the same amount would cost $180,000 per annum. It would thus appear that each municipality could obtain a pure water supply as cheaply by a short intake and filtration as by a long intake pipe, but that a water supply to furnish water to all the district may be most reasonably obtained through a long tunnel. This method must, however, be very carefully considered with reference to the needs of this particular locality. For instance, the long intake without filtration would mean that sewage disposal works would have to be constructed sooner than if a filtered water is to be used.

Modern Procedure in District Sewer Design.

By W. W. HORNER,* C. E.

The writer takes occasion to present this article for a two-fold reason: First, while volumes have been written in recent years on the theoretical side of rainfall and run-off, little has appeared in connection with the detailed computations involved in designing small combined sewers; second, because he hopes to emphasize the importance of a work which has been given far too little attention in the past.

Any one who has observed the peculiar and often expensive ways in which the older sewers of the large cities dispose of the rainfall must realize the importance of this subject, but no one who has not labored over the computations and drawings for a large district can appreciate its complexity. Many engineers still think that this problem can be solved by a progressive application of a formula, and the writer has known several competent men who felt that their intelligence had been insulted when asked to aid in the work. As a matter of fact, the design is as intricate as that for a building or a bridge and has the unsatisfactory side that the "loading" is subject only to the laws of chance.

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Preliminary Layout.

The district usually reaches the designer in the form of a petition for certain sewers; but this can only be taken as expressing a need, for the property holders rarely appreciate the impossibility of sewering a block here and there on request. These requests often come from a community in a valley, where an area farther up the hill must also be included or sewered later through an expensive outlet. More often the requests come from a settlement in the ridge, while the valley is yet unsubdivided. The first work of the engineer, then, is to secure from the larger owners the dedication of streets, or of rights of way for sewers. This is usually a tedious process, but rarely an impossible one; and when impossible, the only alternative lies in condemnation proceedings, and a long delay results. Rights of way are required even when the district is completely subdivided, if the subdivisions have not been laid out with a proper consideration of the topography.

Assuming that all streets and alleys are open and that no rights of way are required, the actual computations can be taken up. The first requisite is an accurate plat (similar to Fig. 1) showing all property lines. On this should be entered the elevations of the proposed or established street and alley grades and, if no contour map has been made, the existing surface elevations should also be shown. The street and alley inlets are then located on the plat, placed on the upper side at all street intersections and at all low points between streets, provided, that the interval so established does not exceed 600 or 700 ft.

An inlet of the type shown on Fig. 2 has a

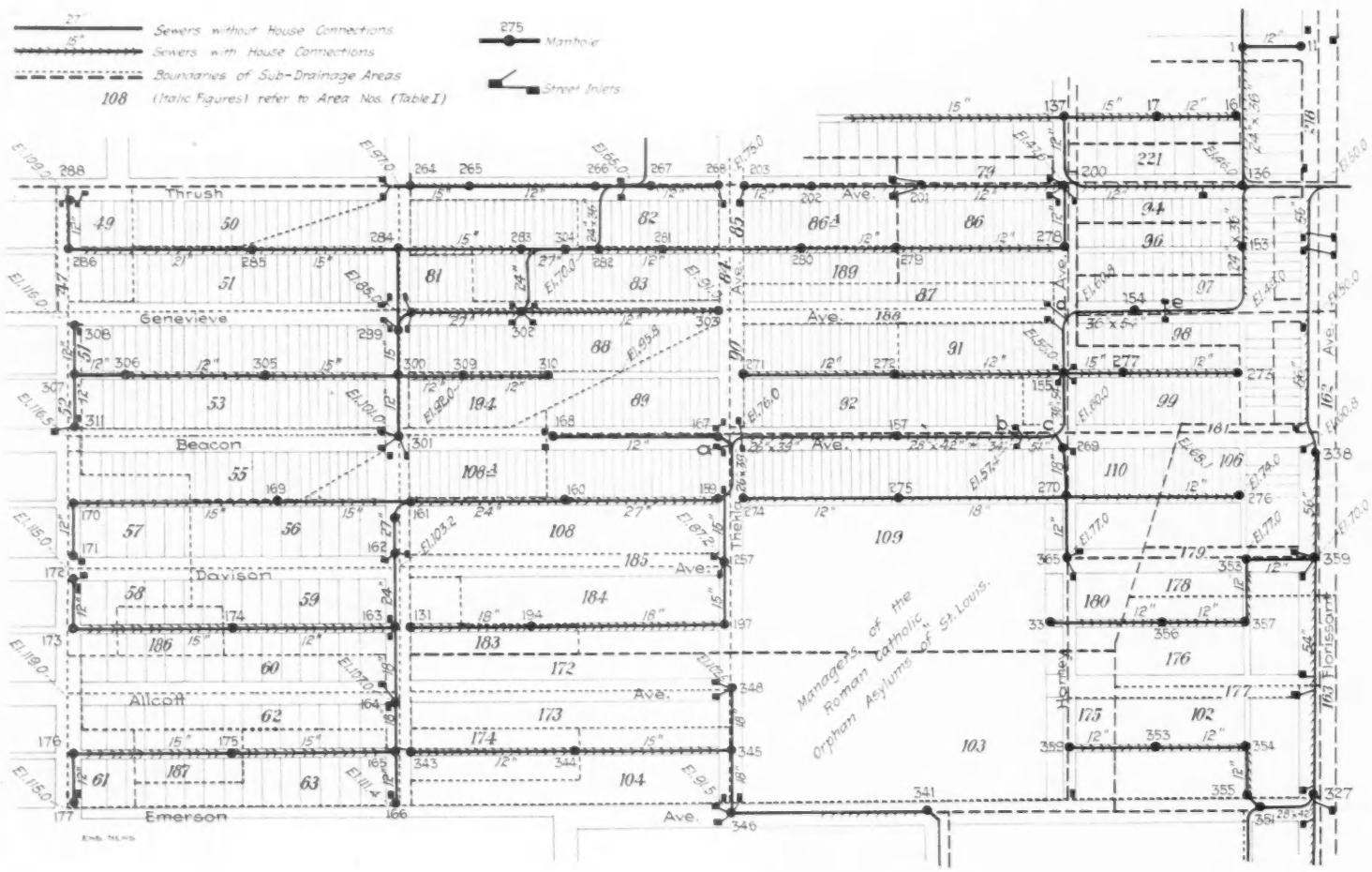

FIG. 1. RECORD PLAT OF DISTRICT SEWERAGE SYSTEM.
(See also Tables I. and II.)

very large capacity if constructed with the proper sump in the gutter and if the outlet pipe is made of sufficient size. The common practice is to make all outlet pipes 12 ins. for a single inlet and 15 ins. where an untrapped basin is attached, but many cases arise where even larger pipes should be used. The writer has seen inlets which satisfactorily drain 1,200 ft. of street, but this is a greater length than is warranted, causing too deep a flow in the gutters.

The inlet should be placed on the upper side of the crossings, as shown on Fig. 3; and, except where in short blocks the pocket in the cross street comes at the alley (Fig. 3), the gutter water should never be allowed to flow around the corner. In cases such as occur at corner (a) in Fig. 3, the low point in the curb should be placed at the inlet, and the water which reaches the gutter between the basin and inlet will flow into the latter. The old form of inlet on the "rounding" has proved unsatisfactory, both on account of choking the gutter at the crossing and because the sump in the gutter interferes with traffic turning the corner.

There are several iron inlets on the market which give good service, but the stone or concrete-topped inlet, shown in Fig. 2, though more expensive, gives a handsomer appearance and is less likely to become stopped. Inlets in alleys are usually required only at pockets between streets, as the surface flow in the alleys is small, and where the grade is toward the street, can be allowed to pass through the gutter to the nearest street inlet. For alley inlets, some type of the various grated covers, on a manhole, has been used extensively but is easily stopped by paper or rubbish. A better practice is to bring the alley to a crown at the low point and place a small iron inlet, similar to the street inlets, but without the trap, on each side.

When the inlets have been "spotted," the next step is to lay out the sewers to reach them, and at the same time to sewer all private lots in the district. Obviously, the most economical layout is one which follows the natural surface slopes in the shortest line toward the outlet of the district, but also concentrates the flow as rapidly as possible. This can usually be done in more than one way, and it is often necessary to make partial designs and comparative estimates to determine which is the cheaper.

The locations of gas, water and other subsurface conduits are next plotted on the map and the alinement of the sewers can then be worked out, nearly in its final form. It will generally be found cheaper to locate the service sewers in the streets, as fewer extra lines are then required to reach the inlets; but, as a rule, in unsewered districts, the house pipes run to vaults on the alley, and the expense to the owners for connections will be less with the sewers in the rear. In addition, in principal thoroughfares, the street is so filled with car tracks and conduits that it may be difficult and expensive to build the sewer among them. Where the sewer is in streets, it is best to lay it in the center, leaving the sides for gas and water, but in some very wide streets (100 ft. and over) it is better to lay a line down each side back of the curb, thus avoiding long connections and damage to the pavement.

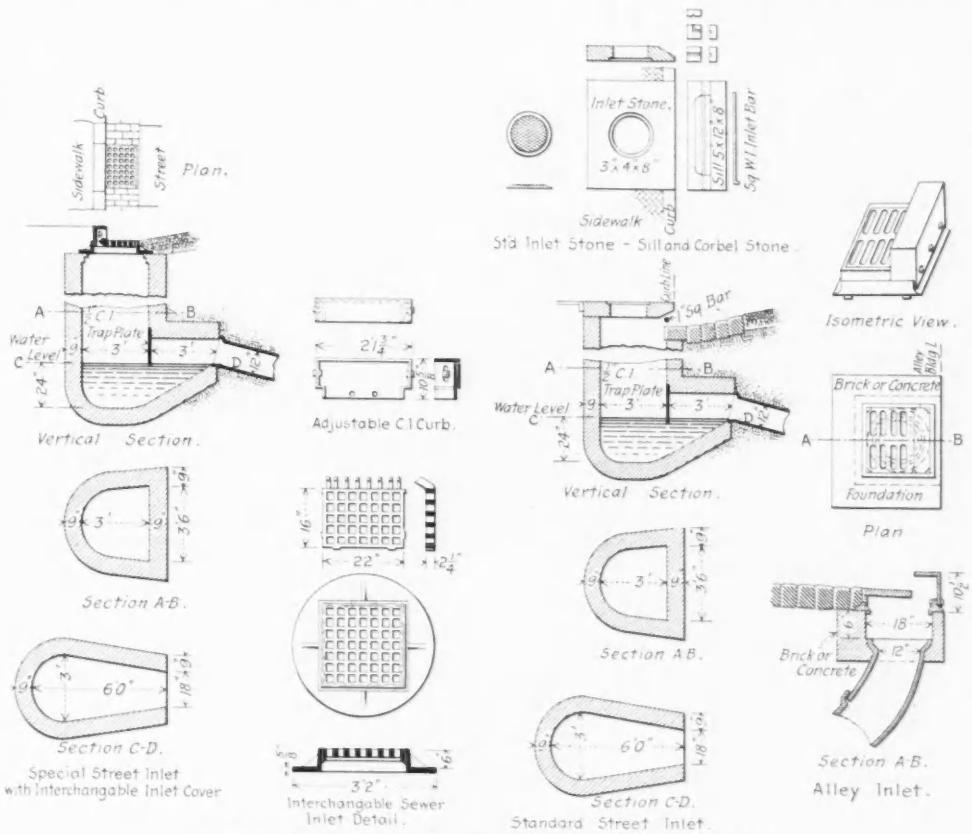

FIG. 2. DETAILS OF SEWER INLETS, ST. LOUIS, MO.

TABLE I.—DRAINAGE AREAS CORRESPOND TO SEWER PLAT SHOWN BY FIG. 1.

Area No.	Area in acres.	Area No.	Area in acres.
56	2.52	97	1.13
57	1.86	98	1.59
58	1.13	99	2.39
59	2.76	107	0.33
60	1.73	108	4.48
61	0.88	108—A	1.45
62	1.78	109	11.15
63	2.12	110	1.91
87	1.47	180	0.87
89	2.11	182	0.24
89—A	0.32	183	1.12
90	0.20	184	3.35
91	1.63	185	0.55
91—A	1.10	186	0.62
92	2.66	187	0.75
96	1.63	188	1.38
	Total = 59.21 acres.		

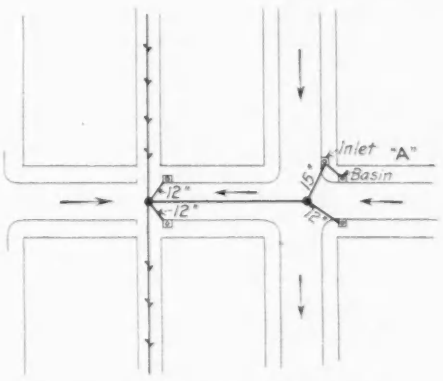

Fig. 3. Location of Sewer Inlets.

With all inlets and sewers located and the present surface elevations on all streets and alleys known, it is a comparatively easy matter to calculate the area tributary to the sewer at any point under existing conditions. This, however, is rarely of great importance in determining the maximum run-off as the district is only usually partly built up and paved, and the percentage of run-off will be small; also much of the surface water will follow old water courses without reaching the sewers.

It is necessary for the designer to stop at this point and imagine the district as it will be when completely settled and paved. Observation of the nearest settled districts and a knowledge of the probable trend of real-estate activity will enable him to estimate what class of property it will be when improved; that is, what class of houses will be built and what amount of grading may be expected in shaping up the lots. In the ordinary flat or small residence districts, the grading will take one of the forms shown in Fig. 4, depending upon the original contour of the ground; in cheaper districts, the original surface will be only roughly smoothed up. In any case, a study of the original surface and of the probable class of improvements will permit the construction of a set of minor ridge lines which will divide the small areas draining into streets from those draining into the alleys; and by these the final areas tributary to each inlet can be shown on the plat and the acreage computed. Fig. 1 shows such a map, and in Table I. are given the drainage area numbers and the area of each, the sum of the areas being checked against the total, computed as a whole. These areas are based on a condition of final grading, but do not take account of the fact that the roof water may or may not flow to the same inlet as the water from the area in which the house is situated; the question of roof water will be considered later.

Profiles and Sewer Grades.

As the alinement has been practically determined from the plat, the stationing for profile work can be computed. The usual method is to start with zero at the outlet of the district, or branch, and follow along the main stem, or longest line, to its end; other lines start with zero at the main line giving the equality at that point.

All brick and concrete sewers are laid out as tangents and curves, the radii of the curves being made as long as possible within the street and alley intersecting. Where curbing is already in place, the curve is kept outside of it, but wherever there is no curb, a clearance of 2 or 3 ft. at the corner is sufficient. Change of direction in pipe sewers should always be made at a manhole; right angle turns may be allowed in 12-in. and even in 15-in. pipe, especially if a small drop is made at the manhole. On larger sizes, and especially on 21-in. and over, the turn should be made by a curve with, if possible, a manhole at each end.

The most convenient form of plan is one in which a plat of the street or alley is shown directly under the profile, making the stationing on the plan, as nearly as possible, a direct projection from the profile.

In addition to the alinement, all water and gas lines, bridges and obstructions can be shown and the location of all manholes and inlets with connections between them. The manholes are numbered both on the profile plats and on the general plat.

The sewer grades should first be drawn in at the proper depth, beginning at the lower end, as the elevation at the outlet is approximately fixed by outside conditions. Then, beginning at the upper end, the final grades can be established at the same time that the sizes are determined. The depths at which the sewers should be laid are subject to so many conditions that it is impossible to lay down a fixed set of rules, but

some points of general application are worthy of mention.

The building conditions will vary in different localities, and those in St. Louis will be used for illustration. Here, such a large proportion of even the cheaper houses are built with cellars that it is necessary to assume the existence of cellars in all cases. In residences, the cellars will average 4 ft. in depth below the front yard —for safety, 5 ft. will be assumed. The yards are usually on a small terrace above the street, but this is not a safe premise, except where the proposed street grade calls for an appreciable cut below the natural surface. It is better to assume the cellar elevation 5 ft. below proposed street grade and to allow 1 ft. for the soil pipe and 2 ft. fall to the sewer. This will make the top of the sewer a standard depth of 8 ft. below street grade, which may be modified where it is certain that the houses will be on terraces. Where stores exist, or are likely to be built, the top of the sewer should not be less than 9½ ft. below grade, to allow for a 7-ft. cellar; and where the natural surface in the back yards is much below proposed grade, it may be necessary to lower the sewer to allow the installation of yard drains at the low points. When the sewers are in alleys, the elevation of the streets on both sides must be considered. Where it is necessary to place lines in adjoining streets and alleys to reach inlets, one line should be laid to sewer the houses, while the other may be shallow, 5 ft. or even slightly less.

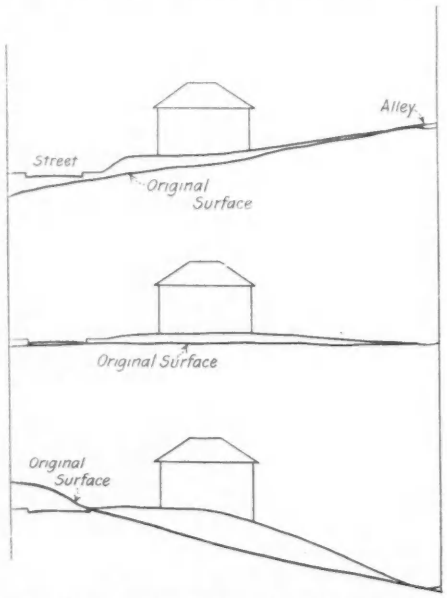

Fig. 4. Types of Lot Grading In Relation to Sewer Depths.

No maximum sewer grade need be considered, if an easy change at the lower end of the line is provided. The writer has seen 12-in. pipe on 10% grade for several blocks; 24-in. on 5%; and 14-ft. sewer on 0.6% and 0.7%, and, in one case 1%—all giving safe operation. In large sewers, however, the inverts should be made of vitrified brick to minimize the loss from sand scouring. On the other hand, grades which give velocities under 2 ft. per sec. should be avoided, and should never be used where there are no inlets at the upper end to flush the sewer.

Where pipes of different sizes meet, the grade line should be stepped up to bring the crown of the two sewers to about the same elevation; or if this is not done, it must be remembered that the real hydraulic grade is not the slope of the sewer, *but is based (both sewers running full) on the difference in elevation between the inside crown of the larger sewer at the junction, and that of the smaller sewer at its upper end.* This point becomes very important where a small pipe sewer discharges into a large trunk, as the outlet will usually be several feet below the water surface line of the trunk sewer when it is flowing full.

Another important point is the compensation of grade for increasing resistance to flow at curves and angles.

It has been the practice in good design to allow an extra drop at manholes wherever the sewer makes any considerable angle; and for some time, also, the writer has been using an arbitrarily increased grade around curves in the larger sewers. Recently, Mr. P. J. Markmann, in a paper before the Engineers' Club of St. Louis, presented a theoretical formula for the "lost head" in curves and for the required grade compensation. This formula is based on the lost work of the "deviating force" in changing the direction of flow, and takes no account of the internal friction and eddy action in the stream; it is, however, theoretically correct and its use will give safe results in all cases. The final formula, as stated, is

$$S_c = S + \frac{v^2}{2gr^2}$$

S_c is the tangent of the slope on the curve S, the slope in the line immediately above the curve, v the mean velocity in cubic feet per second due to S, and R the radius of curvature. As v is dependent upon S and as the total fall depends upon S and S_c, it is impossible to work out the compensation direct, but in placing the preliminary grade line, mentioned above, an experienced designer will allow some extra fall at this point, and one or two trials will give values of S and S_c, which will account for the total allowable fall in the tangent and curve. The computations are simplified by the fact that the value of v is also necessary to the computation of the run-off.

The writer has found several cases where the turn in large pipe sewers is made in a manhole, but this should never be done except in emergency. The manhole will really act as an equalizing chamber, that is, the "velocity head" of flow in the upper pipe will create a pressure head in the manhole causing the water to rise in it to a point at which the pressure is sufficient to force through the lower pipe an amount of water equal to that arriving through the upper; but in order that the upper sewer shall carry the amount for which it is designed, the hydraulic grade must be parallel to the actual grade, and at a height above it equal to the head created in the turn manhole. Thus it is seen that the energy lost in the right-angle turn really acts to raise the hydraulic gradient in the whole system above this manhole, putting all lines under head and causing a back pressure on the house connections.

The writer *knows* that something like this does occur, but has not had the opportunity as yet of making any exact observations. A rough idea of the pressure head may be obtained from the Markmann formula. Assuming a 4-ft. manhole, suppose that the flow makes a smooth turn on a

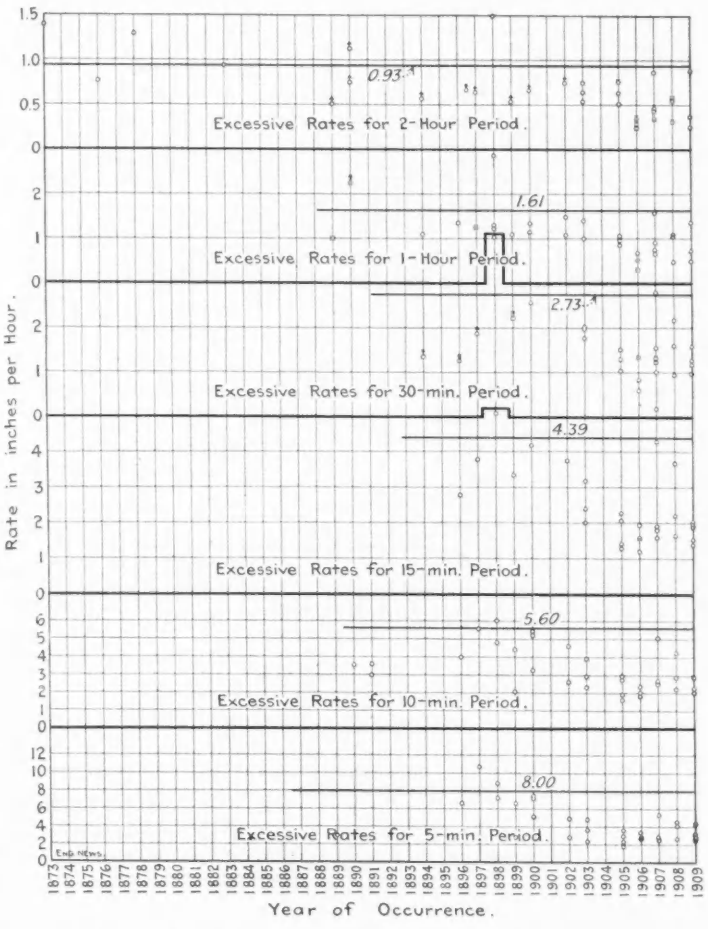

FIG. 5. RATES OF RAINFALL AT ST. LOUIS, MO., FOR VARIOUS PERIODS.
(Data from U. S. Weather Bureau Records.)

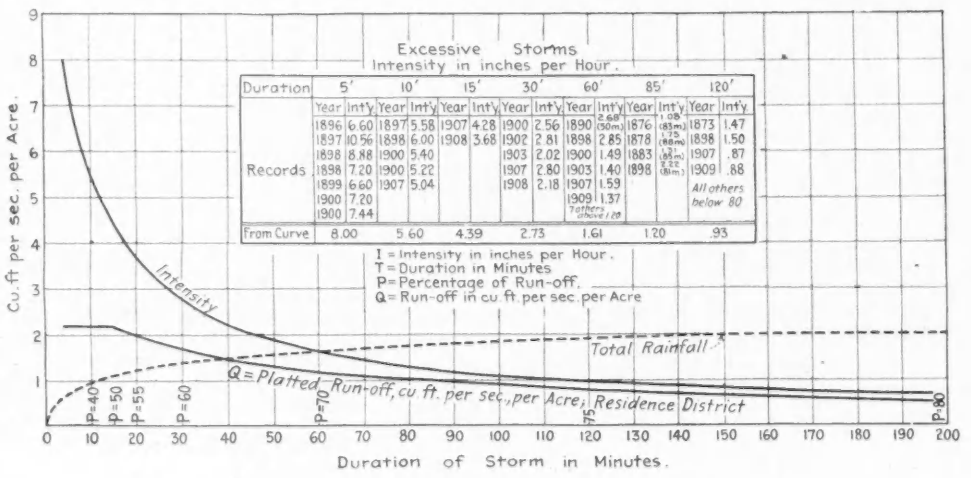
Excessive Storms
Intensity in inches per Hour.
Duration 5′ 10′ 15′ 30′ 60′ 85′ 120′
Records
From Curve 8.00 5.60 4.39 2.73 1.61 1.20 .93
All others below 80
I = Intensity in inches per Hour.
T = Duration in Minutes
P = Percentage of Run-off.
Q = Run-off in cu. ft. per sec. per Acre
Intensity
Total Rainfall
Q = Plotted Run-off, cu ft. per sec., per Acre, Residence District
P=40
P=50
P=55
P=60
P=70
P=75
P=80
Cu. ft per sec. per Acre.
Duration of Storm in Minutes.

FIG. 6. RAINFALL CURVES FOR ST. LOUIS, MO.

2-ft. radius. Then the required compensation becomes $\frac{v^3}{2 \times 32 \times 4} = \frac{v^3}{256}$ and the total fall required is $\frac{v^3}{256} \times \frac{2 \times 3.14 \times 2}{4} = \frac{v^3}{80}$ approximately. Now, for example, a 24-in. pipe on a 1% grade has a theoretical velocity of 7.3 ft. per sec. and the drop at the manhole should be 4.8 ft. (This takes no account of the energy lost in "eddying and boiling" in the manhole.)

Comparatively few such turns occur in the old systems where small brick sewers were used instead of the large pipe; but the writer believes that the small amount of damage from flooded cellars, where the sharp turn has been built, is due entirely to the infrequency of "maximum" storms and to the depth of the sewers, and that in new work where large pipe are coming into general use, this point is a most important one. Instead of the manhole, a pipe curve of long radius should be used, which may be made either of straight pipe or of alternate sections of straight pipe and the longer standard bends.

It is supposed now that the location of all inlets and the alinement and approximate grade of the sewer have been decided upon and that the area tributary to the sewers has been computed in acres. There remains to be determined, the amount of run-off and the size of sewers.

Run-Off.

The capacity for which the sewers must be designed is the product of three primary factors; the area drained, the intensity or rate of rainfall and the percentage of the rain falling which reaches the sewer, or, using the common symbols, $Q = A\ I\ p$. In determining each of these factors, the designer must also be somewhat of a forecaster, but by close study of the conditions he may expect to obtain results which will give a satisfactory system at an economical cost.

The value of A, the area drained, is, for any point in the system, merely a matter of adding up the areas tributary to sewers above that point, and these areas have already been computed.

The second factor, I, is the maximum intensity of precipitation; or, rather, the maximum for which it is economical to build sewers. The value of I varies with the length of time considered, as rates of 10 ins. per hour have been recorded for five minutes during a storm, while for an hour, 3 ins. is high. Practically all of the old empirical formulas, Burkli-Ziegler, McMath, the New York diagrams and others, attempted to allow for this variation by making the rainfall a function of the area drained, but these were, at best, approximate and never gave good values throughout the whole range of acreage. The principles laid down by Kuichling in his Rochester report, and since discussed and refined by Gregory, and carried to the extreme of theorization by Grunsky in papers before the American Society of Civil Engineers, have been generally accepted as the only intelligent theory ever advanced on the subject. The general principle, broadly stated, is that the maximum rate of run-off at any point on the sewer is equal to a coefficient "p," multiplied by the maximum average rate of rainfall for a time equal, nearly, to that required for the water falling on the extreme limit of the area to reach the point in question. At this time, after the beginning of the "downpour," all parts of the area are supposed to be contributing to the flow in the sewer. It is an easy matter to calculate this time of flow without preliminary computation of the sizes, as will be shown, but to formulate the relation between the time and the rate of rainfall, a rainfall curve is required. Such a curve should be computed from the rainfall records for the vicinity in which it is to be used; the method of evolving the curve for St. Louis is given here.

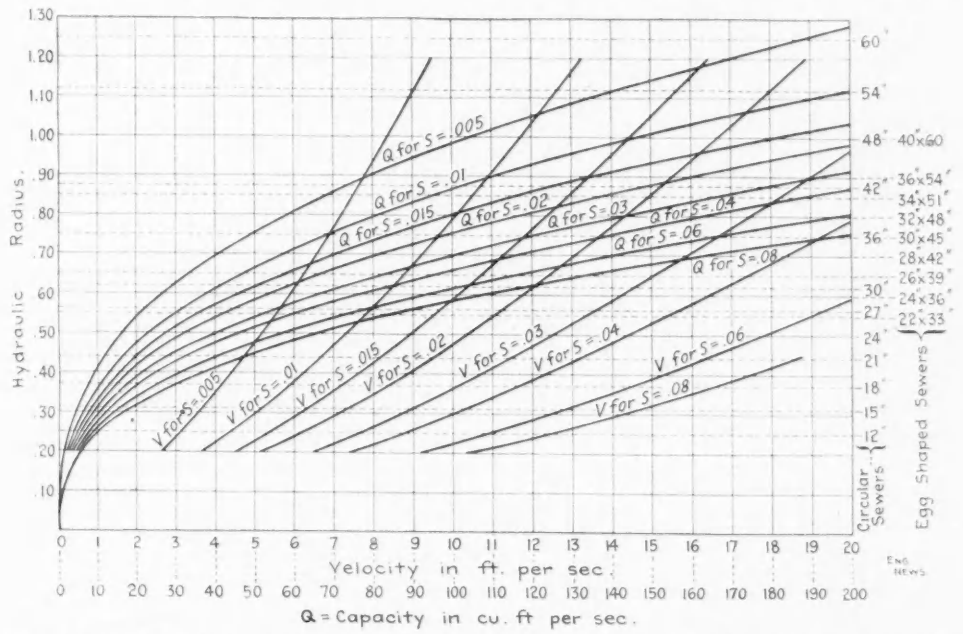

FIG. 7. APPROXIMATE CURVE FOR SEWER DESIGN, ST. LOUIS, MO.; FOR VITRIFIED PIPE AND CIRCULAR OR EGG-SHAPED SEWERS UNDER 60 INS. MEAN DIAMETER.

(Assuming n = .013. Capacity curves correct for both circular and egg-shaped sewers to nearest reading on diagram. Velocity curves correct for circular and about 3% high egg-shaped sewers.)

In Fig. 5 are shown the Weather Bureau records of excessive rains in St. Louis. The abscissae are the years of occurrence and the ordinates are the rates of rainfall. Each point represents a rain; those points in which small arrows are shown indicate that the value was derived from a rain of slightly greater duration, and that the rate shown is therefore, in error by a small amount.

From these graphs, values were chosen for the rainfall rate which excluded storms occurring at greater intervals than about fifteen years.

The values of I were plotted on logarithmic paper with those for t, $t + 5$ and $t + 10$; the second case giving nearly a straight line, with a tangent of 0.85. The resulting formula,

$$I = \frac{56}{(t+5)\ 0.85},$$

has rather an awkward form, but fits the values chosen so closely, that it was retained. The rainfall curve is shown on Fig. 6 as the hyperbolic curve marked "intensity."

The portion of "t" for the flow through the sewers can be calculated, but the portion of the time consumed by the water in travelling over the lawns and streets must, in the present state of our knowledge, be assumed; and, unfortunately, this is rather an important item in small districts. Values from 3 to 15 minutes have been suggested by various authorities, depending upon the character and slope of the surface. The writer, from observation, has reached the conclusion that the water from the streets and sidewalks and roofs will reach the sewer in from 2 to 5 minutes, with street grades of from ½ to 5% (improved streets), but that the velocity over grass plots is very low, and even in heavy rains from 10 to 20 minutes will be required for the water to flow a hundred feet. For the sake of safety, the lower values should be used, and deduction for the lawns made by using a very low percentage of run-off. With this assumption, the total "t" is found and from the rainfall curve the value of I can be taken.

There remains, however, the third factor "p," the percentage of run-off. This factor is not the proportion of the total rainfall which reaches the sewer, a matter of no direct interest to the designer, but represents the relation between the maximum rate of run-off and the average rate of rainfall, during the critical time. Values of the coefficient for the improved districts have been given by various authors, ranging from 15% to over 100%, and can be used only as an indication. Extensive experiments are being made at present in several large cities, St. Louis among them, to determine more accurately the values of "p" under various conditions; but for the present, we are forced to an application of judgment, us-

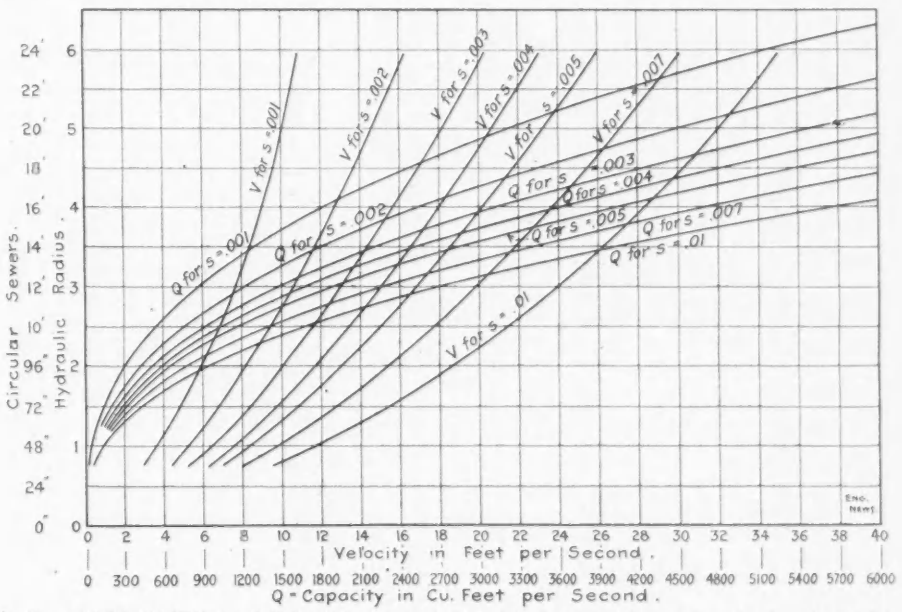

FIG. 8. APPROXIMATE CURVES FOR SEWER DESIGN, ST. LOUIS, MO.; FOR CIRCULAR SEWERS FROM 4 TO 24 FT. IN DIAMETER.

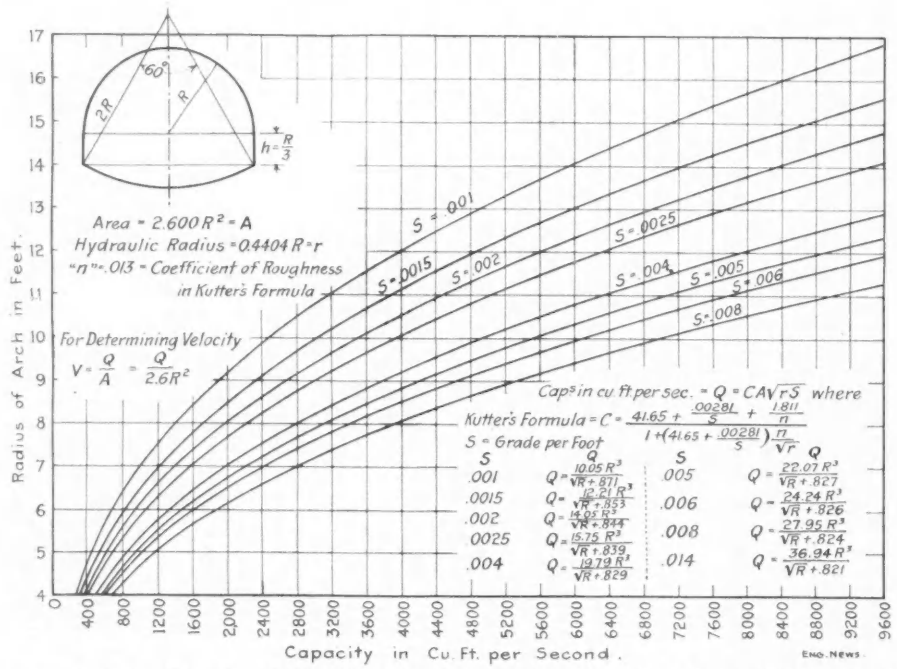
Radius of Arch in Feet.
Capacity in Cu. Ft. per Second.
Area = 2.600 R² = A
Hydraulic Radius = .4404 R = r
"n" = .013 = Coefficient of Roughness in Kutter's Formula
For Determining Velocity
V = Q/A = Q/2.6R²
S = .001
S = .0015
S = .002
S = .0025
S = .004
S = .005
S = .006
S = .008
Cap's in cu ft per sec. = Q = CA√rS where
Kutter's Formula = C = (41.65 + .00281/S + 1.811/n) / (1 + (41.65 + .00281/S) n/√r)
S = Grade per Foot
S | Q
.001 | Q = 10.05 R³ / √R + .871
.0015 | Q = 12.21 R³ / √R + .853
.002 | Q = 14.03 R³ / √R + .844
.0025 | Q = 15.75 R³ / √R + .839
.004 | Q = 19.79 R³ / √R + .829
.005 | Q = 22.07 R³ / √R + .827
.006 | Q = 24.24 R³ / √R + .826
.008 | Q = 27.95 R³ / √R + .824
.014 | Q = 36.94 R³ / √R + .821
Eng. News

FIG. 9. CAPACITIES OF STANDARD HORSE-SHOE SEWER: ST. LOUIS SEWER DEPARTMENT.

ing the recorded values for what they may be worth. The derivation of the values for St. Louis is given here and may be used as a guide for other localities. An average half city block in the newer subdivisions is assumed. This one-half block from center of street to center of alley will be 172½ ft., and from center to center of cross streets, 860 ft., giving a total area of 3.38 acres. The impervious portion of this block will be approximately:

	Sq. ft.
Streets	20,000
Alleys	6,500
Sidewalks	6,500
House roofs	27,000
Shed roofs	4,000
Yard walks	2,000

This makes a total of 1.53 acres. The percentage of the total area, which is entirely impervious, will then be about 45 (the population for such a block runs about 40 per acre.)

For rain of ten minutes duration, it was assumed that 60% of the water falling on impervious surface, and 20% of the water falling on lawns and hard ground, would run off. This gives an average of 38% run-off for the whole block.

The following table gives the assumed percentages for each class, and the final average value of p in round numbers:

Duration "t."	Per cent. run-off from Impervious portion.	Per cent. run-off from Pervious portion.	Value "p."
10	60	20	.40
15	70	30	.50
20	80	35	.55
30	85	40	.60
60	95	50	.70
120	95	60	.75

The writer believes that accurate data, which experiments now under way will give, will establish slightly lower values for the short times, and greater values for the longer periods.

Now, multiplying the intensity in inches per hour, or in cubic feet per second per acre, by these percentages, the lower curve in Fig. 5 was obtained. This curve gives directly the discharge in cubic feet per second per acre for any critical time of duration, but applies only to the average residence district for which the percentages were derived.

The curves on Figs. 7, 8 and 9 are graphical representations of Kutter's formula and are given here to facilitate the computation of the time interval. The value of "n" is taken as .013 in each case, and is probably a little high for the vitrified pipe. All three figures are calculated for sewers running full, as there is no doubt that the "60° free arc" maximum is a condition which is not obtainable in practice. Fig. 7 applies to vitrified pipe and circular or egg-shaped brick sewers under 60 ins. mean diameter, and Fig. 8 to circular sewers from 4 to 24 ft. in diameter.

The use of these curves is as follows: When the area above the first inlet has been determined and the time decided upon, the total discharge at this point is obtained by multiplying the area in acres by the discharge in cubic feet per second per acre, corresponding to the time assumed. With this discharge enter the graph of Fig. 7 on the horizontal line marked "Capacity cubic feet per second," follow the vertical line through the given value of the discharge to its intersection with the line representing the grade of the sewer to be designed. Horizontally to the left of this point will be found the size of sewer required, and moving horizontally to the right to the velocity curve corresponding to the same grade, thence vertically to the base line, the velocity in this section will be found; dividing the length of the section by this velocity, the time of flow through this section is obtained, and adding this time to the time at the upper end, the critical time for the design of the next section is found. This system lends itself readily to a condensed form of record, an exhibit of which is given here in Table II.

In Table II. the sewer lines are denoted by the number of the manholes at the ends. The drainage areas are also given by number, both being shown on a record plat, Fig. 1, which accompanies the set of tables.

The column A^1 gives the area tributary to the

TABLE II.—DATA FOR BRANCH SEWER (see Fig. 1).

Line M. H.	to M. H.	Drainage Area	A′	Acres	S in %	v	l	t	Σ′	pI	Q	Size
*177	176	61	.88	.88	.50	3.10	125	.70	7.00	2.20	1.94	12″ —
*176	175		.88	1.67	.50	3.60	415	1.90	7.70	2.20	3.68	15″ —
*175	165	+ 187	1.63	3.15	1.00	5.20	410	1.30	9.60	2.20	6.93	15″ +
166	165	63	2.12	2.12	1.50	5.30	125	.40	8.00	2.20	4.67	12″ +
		M.H.										
*165	164	(165)	3.75	4.30	1.50	7.20	130	.30	10.90	2.20	9.46	18″ —
*164	163	+ 60 + 62	7.26	7.26	3.00	10.20	195	.30	11.20	2.20	15.98	18″ —
172	173	58	1.13	1.13	.50	3.10	125	.70	6.00	2.20	2.49	12″
173	174		1.13	1.82	.50	3.60	425	2.00	6.70	2.20	4.01	15″ —
174	163	+ 186	1.75	3.09	3.00	7.60	400	.90	8.70	2.20	6.80	12″ +
		M.H.										
*163	162	(163) + 59 + 107	12.10	12.10	1.00	7.30	195	.40	11.50	2.20	26.64	24″ +
*162	161	+ 56 + 182	14.86	14.86	1.00	7.90	130	.30	11.90	2.20	32.70	27″ +
171	170	57	1.86	1.86	1.50	5.30	120	.40	8.00	2.20	4.09	12″ —
170	169		1.86	2.62	.75	4.10	510	2.10	8.40	2.20	5.77	15″
169	161		1.86	3.41	2.00	7.30	310	.70	10.50	2.20	7.50	15″ —
		M.H.										
*161	160	(161)	16.72	18.72	3.25	12.80	435	.60	12.20	2.20	41.20	24″
160	159		16.72	19.83	2.50	12.20	415	.60	12.80	2.20	43.65	27″ —
191	194		.00	1.10	2.00	6.10	310	.80	5.00	2.20	2.42	12″ —
194	197	183	1.12	2.92	.60	4.40	500	1.90	5.80	2.20	6.43	18″ —
197	257		1.12	2.92	2.00	7.30	160	.40	7.70	2.20	6.43	15″ —
257	159	+ 184 + 185	5.02	5.57	5.00	11.20	165	.20	8.10	2.20	12.25	15″ —
		M.H.										
*159	a	(159) + 108	26.22	28.15	2.00	12.40	185	.20	13.40	2.20	61.95	26″ × 39″ —
168	167	89A + 108A	1.77	2.38	4.75	9.30	440	.80	7.00	2.20	5.24	12″ —
167	a	+ 89 + 108	8.36‡	8.36	29.30	....	40	..	7.80	2.20	18.40	12″ —
*a	157	(a)‡ + 90	30.30	32.85	2.56	13.50	400	.50	13.60	2.20	72.30	26″ × 39″ —
*157	b		30.30	33.47	1.50	11.30	225	.30	14.10	2.20	73.75	28″ × 42″ +
*b	c	+ 92 + 109°	44.11	45.67	1.00	10.50	220	.40	14.40	2.20	100.50	34″ × 51″ +
274	275		.00	1.12	1.50	5.30	405	1.30	5.00	2.20	2.47	12″ —
275	270		3.00†	4.10	.80	5.30	435	1.40	6.30	2.20	9.02	18″
365	270	180	.87	.87	2.00	6.10	165	.50	5.00	2.20	1.91	12″ —
276	270		.00	1.10	1.25	4.70	420	1.50	5.00	2.20	2.42	12″ —
270	269	(270)† + 109°	12.02	13.12	6.00	14.30	125	.10	10.00	2.20	28.88	18″
269	c	+ 110	13.93	14.48	16.70	....	40	.10	10.10	2.20	31.85	18″ —
*c	155	(c) ‡ † °	46.89	48.89	1.00	10.10	125	.20	14.80	2.15	105.00	36″ × 54″ —
271	272		.00	.55	2.50	6.70	400	1.00	5.00	2.20	1.21	12″ —
272	155	188	1.38	2.14	4.75	9.50	440	.80	6.00	2.20	4.71	12″ —
273	277		.00	.86	1.50	5.30	310	1.00	5.00	2.20	1.89	12″ —
277	155	99	2.39	2.69	1.00	5.20	150	.50	6.00	2.20	5.92	15″ —
*155	d	(155) + 91	52.29	54.59	1.00	10.90	150	.20	15.00	2.15	117.20	36″ × 54″ +
*d	154	+ 87 + 91A	54.86	57.55	1.00	10.90	175	.30	15.20	2.14	123.10	36″ × 54″ +
*154	e		54.86	57.94	1.00	10.90	140	.20	15.50	2.14	124.00	36″ × 54″ +
*e	153	+ 97 + 98	57.58	60.06	1.00	10.90	300	.40	15.70	2.14	128.60	36″ × 54″ +
*153	Main Line	+ 96	59.21	61.41	1.00‡	10.90	160	.20	16.10 16.30	2.10	129.00	36″ × 54″ +

°Drainage area No. 109 has been figured as entering the inlet both at Harney and Beacon Aves., and also at Harney Ave. and the alley south of Beacon Ave.
†3.00 acres of drainage area No. 109 has been figured as entering at Manhole No. 275.
‡Drainage area No. 108 has been figured as entering the inlet both at Thekla and Beacon Aves., and also at Thekla Ave. and the alley south of Beacon Ave.
The letters used in the column headings are explained in the text of the article.
*Shows the main stem of the district.

sewer when there are no house connections, and the column A shows how the water reaches the sewer when roofs are taken into account. This latter is almost invariably the greater as the water from the roofs reaches the sewer higher up than that from the inlets. Many cases occur also where the roof water goes into one branch and the surface water into another, so that the totals for A and A^1 are not always equal.

The allowance for roof water is obtained by assuming 80% of the block length to be house frontage, the depth of the houses being, for safety, taken as 45 ft. This gives 36 sq. ft. per foot of frontage. But the run-off from the roofs is to be taken as 80%, while that from the average block for a 15-minute interval will be about 50%. We must, therefore, increase the 36 sq. ft. to an equivalent area at 50%, which will give the same run-off. That is, $\frac{80}{50} \times 36 = 58$ sq. ft., or .00133 acres per front foot. This can be added in with the other surface areas, remembering that in the general half block the roofs having already been accounted for and that this allowance is only to be used where the roofs alone drain to the sewer, the surface water going to another line. When the two sewers meet, the values in the A column should be added and those in the A^1 column should be dropped.

Materials.

In the older sewers, vitrified pipe up to 21-in. were used, and where larger sizes were required brick sewers were built. At present, however, 24-in. and 27-in. are in general use, and many cities are putting in 30, 33, 36 and 42-in. vitrified pipe. Against these latter, a strong prejudice still exists, and where they are not used, the standard egg-shaped brick sewer, having a height of 3/2 of the width, is by far the best section. The St. Louis standards provide for its use up to the 30 × 45-in. section in all cases, and in good ground up to the 40 × 60-in. The long, flat radius of the sides, however, make the sewer weak against horizontal pressure, and the larger sizes have, from experience, been found unsafe in soft ground. All egg-shaped sewers and the circular sewers under 54 ins. diameter, are built of two-ring brickwork, except in special cases. Circular brick sewers of three-ring work are used up to about 9 ft. diameter, except in deep rock cuts, where special sections are designed for each case. Sewers larger than the 9 ft. circular, are made of horseshoe or semi-elliptic shape and are built of reinforced concrete, according to standards which have been worked up, but which would be too complicated to consider here.

The point on which designers are most likely to disagree, is in relation to the use of large vitrified pipe. As has been stated, the construction of pipe sewers in sizes over 24-ins. is regarded as somewhat of an experiment, a feeling with which the writer is not in entire accord.

St. Louis had an unpleasant experience with the failure of 21 and 24-in. pipe about eight years ago, and the remembrance of this trouble has prevented the use of the very large sizes; 27-in. pipe, however, are now used quite generally. The writer believes that with proper specifications and testing, and with careful laying, vitrified pipe up to 42-in. will give safe and satisfactory sewers. The cost, at current prices, is slightly in favor of the pipe for 27-in., compared to 24 × 36-in. brick (about $2 against $2.20 per ft., including junctions). For larger sizes, the pipe will be slightly more expensive. The writer would prefer to use vitrified pipe up to 42-in. in dry ground where little bracing is required, but in bad ground the brick sewer is better and cheaper.

Angular Loading-Track Arrangement for Panama R. R. Docks at Cristobal.

By E. B. KARNOPP.*

The study of a track layout most suitable for serving the proposed new dock of the Panama R. R. at Cristobal, the Atlantic terminus of the Panama Canal, showed that the position of loading tracks can affect the cost of handling freight between ships and cars about 20%. The cargo to be handled here is package freight, i. e., general merchandise in boxes and packages, which is unloaded from the ship by tackle or skids and then wheeled into cars by means of the ordinary hand-trucks. Special docks with handling plant and cantilever crane are reserved for coal, ties, rail and piles, and solid ship-loads of supplies such as dynamite and cement.

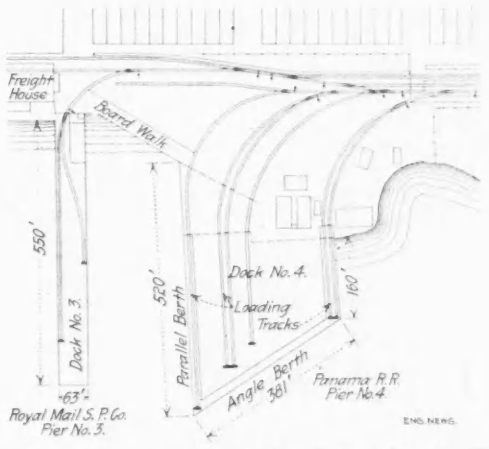

Fig. 1. Panama R. R. Dock No. 4, Cristobal, Canal Zone, Panama.

(Arranged for testing comparative efficiency of parallel and angular loading tracks.)

The proper position of loading tracks is at an angle to the ship and not parallel to it, as is customary. Panama Railroad Dock No. 4, Cristobal, Canal Zone, as shown in Fig. 1, illustrates both positions. When a ship is at the side toward the open sea marked "Angle Berth," the loading tracks are at an angle of about 60° with the ship, but when a vessel is in the slip between docks No. 3 and No. 4, marked "Parallel Berth," then these same tracks lie parallel to the ship. The relative merits of both types of tracks under the same conditions have been compared here. The accompanying table shows data collected by one of the steamship companies, giving the cost per ton for "stevedoring" for a period of three months. Stevedoring means, in this case, transferring the cargo from the ship's hold onto the dock floor. The packages are assembled into rope slings, hoisted by the ship's tackle and deposited on the floor in complete consignments, after which the railroad company does the "handling," that is, transfers the cargo from the ship's side to the cars.

COST OF STEVEDORING AT CRISTOBAL DOCK OF PANAMA R. R.

Ships at Angle Berth				Ships at Parallel Berth			
3,700	tons	@ 11½¢	$425.50	900	tons	@ 20¢	$180.00
4,950	"	" 12¾¢	631.12	780	"	" 18¢	140.40
3,400	"	" 17½¢	595.00	900	"	" 17½¢	157.50
950	"	" 18¾¢	178.13	700	"	" 19¼¢	134.75
950	"	" 15¼¢	144.88	1,170	"	" 17¢	198.90
1,200	"	" 13½¢	162.00	940	"	" 18¢	169.20
4,650	"	" 16¾¢	755.63	655	"	" 25¢	163.75
				800	"	" 20¢	160.00
				1,000	"	" 12¢	120.00
				800	"	" 20¢	160.00
				730	"	" 16¢	116.80
19,800	tons	@ 14.6¢	$2,892.26	9,375	tons	@18.1¢	$1,701.30

The figures in the table show for this one company's business, a reduction in cost of 3.5 cts. per ton, or 18.9% in favor of the angle tracks, and although the average cargo per ship while at the angle berth happened to be three times that of the parallel berth, yet these figures represent typical conditions.

The same relative economy holds true for the railroad's "handling." It has been observed that an amount of cargo requiring, at the parallel berth, about four days to be transferred to cars, can be cleared away in about three days when at the angle berth.

The angle tracks have several advantages not possessed by the other arrangement. When freight is stored on the dock floor, as may happen from shortage of cars or labor, it can be scattered over a great area where it becomes more accessible and does not have to be piled up on a small platform space, which evidently, under the parallel track scheme, would have to be narrow to make the trucking distance between the ship and cars as short as possible. It is a fact that it is much cheaper to truck freight several hundred feet than to pile it up in stacks. Another advantage is that any track can be switched without interfering with the others and there need be no long line of filled cars waiting for a few to be completed.

Inbound and outbound freight can be worked at the same time, which is not feasible with the parallel arrangement. The cars on the second parallel track can only be loaded by trucking through the cars on the track nearest the ship, but every car on an angle track can be loaded without interfering with another.

Fig. 2 shows the plan of the proposed Dock No. 12 as adopted. The tracks are in pairs at an angle of 45° with the ship, but would have been made 90° if more ground space were available. All the tracks are depressed so that the floor of the dock is flush with the cars. The southern end of Dock No. 11 is also shown at the left in Fig. 2. It has been in operation several years with two parallel loading tracks. The width of floor and roof is 110 ft. from face line to the back wall. The width of the proposed dock is 207½ ft.

The angle arrangement of loading tracks requires greater area of ground, superstructure and floor, but its installation is to be recommended when the saving in operation will more than offset the additional cost of construction.

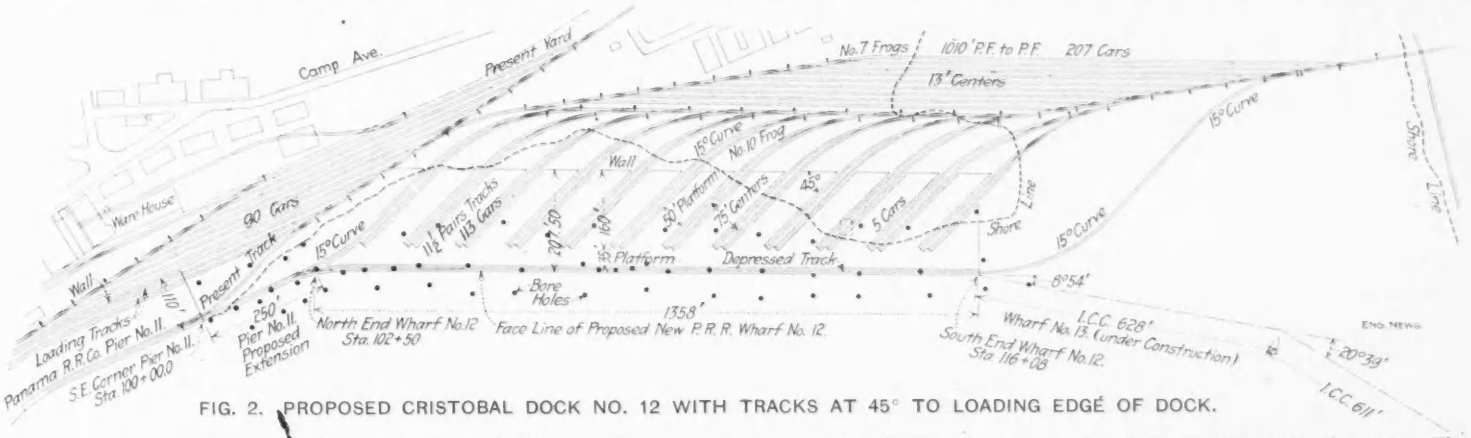

FIG. 2. PROPOSED CRISTOBAL DOCK NO. 12 WITH TRACKS AT 45° TO LOADING EDGE OF DOCK.

*Assistant Engineer, Panama R. R., Cristobal, Canal Zone, Panama.

A New Recording Loss-of-Head Gage for Sand Filters.

By F. B. LEOPOLD.*

The first recording loss-of-head gages placed on any filter plant, to the writer's knowledge, were put in operation at Lorain, Ohio, in 1906. The record consisted of a circular chart on which was recorded the loss of head on the effluent to the filters. The working of these gages was very satisfactory in that a record of the operation of the bed was obtained from which considerable valuable information was derived, and they were used on several later plants. However, when the

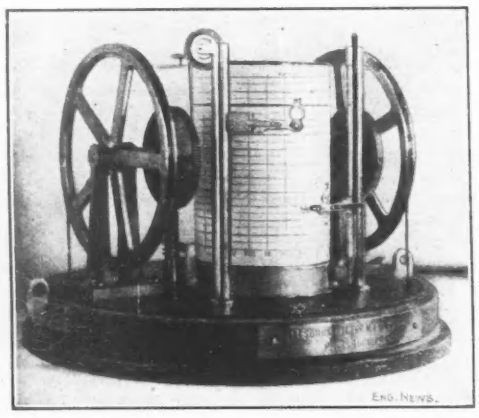

Fig. 1. Recording Loss-of-Head Gage for Sand Filters.
(Glass case removed to give clearer view of mechanism.)

gages were applied to a plant with considerable variation of water level above the sand bed, it was at once demonstrated that, to make the readings of any value, a uniform level must be maintained, and this experience led to the development of a type of gage which gives the actual loss of head irrespective of variation of water level above the sand bed.

This gage is entirely different from anything that has heretofore been used in the operation of filters. It comprises both a mechanism for recording the water level carried over the sand bed of the filter and a second mechanism which records the loss of head shown in the effluent from the filter. The ordinates between the two recorded lines gives the absolute loss of head in passing through the filters.

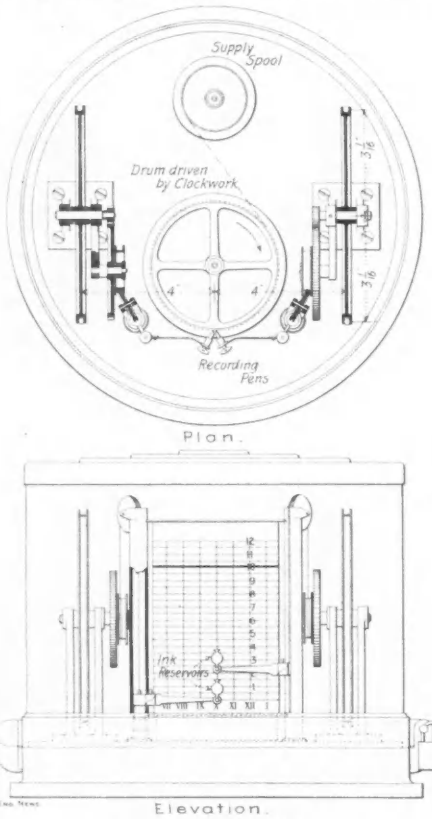

Fig. 2. Details of Recording Mechanism of New Loss-of-Head Gage.

*General Manager, Pittsburg Filter Manufacturing Co., Farmers' Bank Bldg., Pittsburg, Pa.

RECORDING MECHANISM.—The general appearance of the instrument is shown in Fig. 1, and its construction is made clear by the accompanying drawing, Fig. 2. In use, the gage is mounted in a cylindrical glass case (not shown in Fig. 1) about 12 ins. in diameter and about 12 ins. high. The pen of each of the two recording mechanisms is carried by a traveler which is guided in a hollow vertical column. Thus the pens move in a true vertical line instead of along the arc of a circle as in the usual arrangement. The travelers carrying the pens are attached to cords winding on drums which are actuated, through reduction gearing, by another system of cords and pulleys communicating directly with the two floats in the filter water.

FLOAT MECHANISM.—The floats for operating the pens are of the ordinary copper-ball type carried in float tubes, in which they rise or fall with the level of the water in the tubes. One of the float tubes is connected into the water above the sand bed of the filter, while the other tube communicates with the lowest point of the effluent from the filter at a point just previous to the entrance to the controller. The motion of the floats is transmitted by cords to the 6-in. pulley wheels shown plainly in Figs. 1 and 2.

Before the filter bed is first placed in operation, the water level in the two float tubes is, of course, the same. When the plant is started, however, the level in the effluent tube recedes to a point corresponding to the initial friction in the sand bed. As the bed becomes gradually

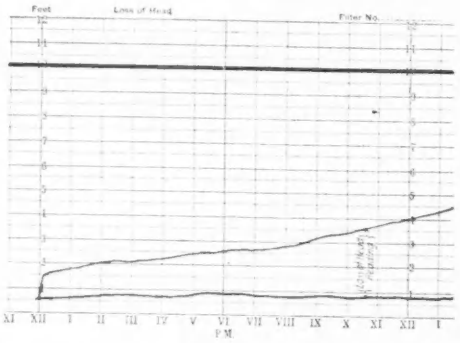

Fig. 3. Specimen Chart and Record Made by Loss-of-Head Gage.

clogged up, the hydraulic friction through the sand bed increases and the water in the effluent tube is correspondingly lowered until a full loss of head is indicated.

FORM OF RECORD.—The form of chart and a specimen record is shown in Fig. 3. The reduction gearing between the float drum and the pen drum is so designed that the vertical readings on the actual chart are at a scale of ⅜-in. to the foot. The scale of the horizontal readings is ½-in. to the hour. This gives a chart for one day (24 hours) rectangular in shape, 6 ins. × 12 ins.

The gages are fitted with seven-day clock movements, covering a week's operation. The chart on removal may be clipped into the separate days, each dated and filed. This makes a most convenient method of filing, as the charts may be bound in an elastic binding in book form, so that they are readily accessible for reference at any time—a great improvement in convenience over the ordinary circular form of chart. Furthermore, the gages give an immediate and direct reading of the working of the filter bed without the necessity of charting curves afterwards. By the use of the two pens a record of the absolute condition of the bed is made available at all times, a matter of the utmost importance in the operation of a filter plant, if detection of irregularities is desired.

Gages of the type described, made by the Pittsburg Manufacturing Co., are now being installed in several filtration plants under construction. Application has been made for patents covering broadly the essential features of the operating and recording devices.

The Development of Rails for Street Railways.*

E. B. ENTWISLE.†

Before electric traction became general, and even for a limited period thereafter, the use of the stringer form of rail was almost universal. These rails were of the side and center bearing types. They were spiked to wooden stringers which rested upon ties spaced 7 to 10 ft. apart.

As the old form of rail proved unequal to the demands of heavier equipment, the girder rail was developed. The first girder rail was rolled by the Cambria Iron Co. This rail was 3 ins. high and wide, weighed about 35 lbs. per yd., and had a tram ¾-in. wide. It was used on one of the cable railways in San Francisco, and rendered good service for over 20 years. Although this was rolled in 1877, this exact form was patented in 1885 by Elizur Barnes. The narrow tram feature is specifically set forth in the description as being "wide enough to clear the flanges of car wheels, but not wide enough for carriage wheels." However, prior to either of these dates, we have record of a United States patent granted to S. A. Beers in May, 1859, which in all essential particulars is the prototype of the common form of girder rail of to-day.

From 1885 to 1890 various forms of girder rail were proposed, many being patented. The Richards rail of 1887 was one of the earlier forms of grooved rail, of which large quantities were rolled by the Johnson Co. This was one of the first to have a semblance of a lower flange, the present mode of rolling grooved and Trilby rails on a universal mill not having been attempted at that time.

In order to obtain the advantage of a groove, or a guard such as we now find necessary on short radius curves, the ordinary form of girder rail was used and the guard formed by rivetting a bar of rectangular shape to the tram. As the demand for a more stable construction increased and the facilities for rolling advanced, the present form of guard rail was produced, which unites the guard to the head in a solidly rolled section with full lower flange. This form of rail was not successfully produced until 1892, and then only in sections 4½ ins. high.

The necessity for rails of greater height than 4½ ins., in order to take care of brick or stone block paving and yet keep the weight of rails under 60 lbs. per yd., very early led to the use of supporting chairs. Those were much used in tracks laid in the '80s. They were drop forged, generally 4 ins. high, and were in quite common use until rails 6 ins. (and later 9 ins.) high were rolled. The use of the higher rails has caused the almost entire disappearance of chair construction. The 9-in. girder rails were first rolled late in 1892, and about two years later the 9-in. guard rail followed.

In 1890 the Gibbon rail was introduced, and one railway at least tried the experiment of operating electric cars over this form of construction. It consisted of the old form of stringer rail with a small depending flange through which it was secured to short cast-iron boxes placed at about 5 ft. intervals. No cross ties were placed under the boxes and the rails were simply held to gage by tie-bars. This proved to be unsuccessful and the same inventor later produced the Duplex rail which was in two halves, one representing the head, the other the tram. These broke joint about midway in each length and were secured through the vertical webs to cast-iron chairs which in turn were spiked to cross ties in the usual manner. So far as the writer is informed this is the only rail rolled in two distinct halves which ever secured an actual trial in track. Many other forms of duplex rails have been patented and are still being invented, but their alleged advantages do not appear to have induced many engineers to place them in use.

A form of box girder rail was patented by W. C. Wood in 1890, and known as the Lewis & Fowler rail. This form of rail had many strong adherents and was for a time commercially successful. But of late years it has given way to the present single-web girder rail, which is now considered to be the standard, as upwards of 870,000 tons of girder rails have been produced by one firm alone within the last ten years.

In the 25 years in which the Lorain Steel Co. and its predecessor the Johnson Co. have been rolling girder rails, 200 different sections have been produced. These have varied from 3½ to 9 ins. in height, and from 30 to 164 lbs. per yd. About half of these sections are

*From a paper on "Track Work" presented at the meeting of the Central Electric Railway Association at Indianapolis, Ind., Sept. 22, 1910. The paper covered the history of steel rails from the old strap rail of 1716. We have abstracted simply the part relating to street railway track.—Ed.

†Chief Engineer, Lorain Steel Co., Johnstown, Pa.

now available, the remainder being declared obsolete. The Chicago street railways are now using a girder rail of the Trilby pattern 9 ins. high, and weighing 129 lbs. per yd. The first rail of the new construction was laid June 18, 1907.

The necessity for a girder rail weighing 129 lbs. per yd. is questionable. However this may be, manufacturers are willing to roll lighter or even heavier rails provided engineers call for them. It must be remembered that car builders have ever kept to the fore with larger cars, and the electrical men with more powerful and heavier equipments.

It may be that we will have still larger cars as the necessity for transporting increasing numbers occur, and that larger and heavier rails will be required. But to the writer it appears that for the width of street set apart for the use of street railways in the majority of our cities, we have about reached the limit in width (if not in length) of the cars, and perhaps in the weight of rail to carry them at the minimum cost of maintenance. Our efforts now should be directed towards the standardization of rolling stock and track, that economy in first cost as well as maintenance may be successfully practised. The work now being prosecuted by the standardization committees of the American Street & Interurban Railway Association deserves our warmest commendation as being a step in the direction of reducing cost to the manufacturer as well as the user of street railway materials.

Annual Convention of the New England Water-Works Association.

The 29th annual convention of the New England Water Works Association was held at Rochester, N. Y., Sept. 21-23, 1910. Combined with the convention was an outing trip, beginning for the New England members at Boston Monday evening and ending in the same city the next Sunday morning. Those who joined in this outing reached Rochester Tuesday morning and so had a full day in that city before the convention opened. A second free day was had in Rochester on Friday, followed by a trip to Niagara Falls so arranged as to give the whole of Saturday there. A considerable portion of Wednesday was also available for seeing Rochester, on the afternoon of which day automobiles were put at the disposal of the members and guests for a ride of some 30 miles through the streets and parks of the city, including a visit to the new Cobb's Hill Reservoir.

The attendance at the convention numbered something over 200, divided in approximate equality between members, associate members and guests. An unusually good exhibit of water-works appliances was participated in by some 25 associate members and about half that number of active members.

By combining two sessions for the presentation of papers into one, the total number of sessions devoted to the presentation and discussion of water-works subjects was limited to four. Notwithstanding this, the proceedings were so well conducted, and the authors so modest in their demands upon the time of the convention, that a considerable number of unusually interesting and valuable papers was presented, and there was plenty of time for discussion.

The Water-Supply and Parks of Rochester.

The members and guests, having been afforded an opportunity to see the streets and parks and a portion of the water-works of Rochester during the automobile trip on Wednesday afternoon, were well prepared to hear the interesting series of papers descriptive of the public works of Rochester presented on Wednesday evening. The first of these papers was by Mr. E. A. Fisher, City Engineer of Rochester, who gave a general description of the city and its growth in population, assessed valuation, taxation and the like, not forgetting to give a good general idea of the framework of the city government. As to the latter, it may be noted that Rochester has a relatively small city council, which passes local ordinances and appropriation bills, on the one hand, and a mayor and various other executive officers, on the other, the latter being appointed by and responsible to the mayor. A board of estimate and apportionment frames the city budget, which may be cut down, but cannot be increased, by the city council. All contracts above a few hundred dollars in amount are let by the board of contracts and awards. The city engineer is a member of each of these boards.

A good general idea of the water-works system of Rochester, together with details of some of its most interesting features, was presented by Messrs. Frederick T. Elwood, Commissioner of Public Works; Beekman C. Little, Superintendent of Water-Works, and Mr. John F. Skinner, Principal Assistant Engineer. As many of our readers know, Rochester was the first city in the United States to introduce dual water-supply, embracing a full supply along the usual lines and an independent supplementary supply for fire protection and for manufacturing uses in the business section of the city. The main supply comes by gravity from Hemlock Lake, some distance north of the city. The special supply is pumped from the Genesee River, within the city limits, which, even in the early '70's, when the works were built, was already too badly polluted to be suitable for domestic consumption. Connected with this special supply is a Holly quadruplex pump which has now seen over 35 years of service; also modern installations of centrifugal pumps, one driven electrically and another by a steam turbine. The main supply has long been notable because (1) the gravity supply main from Hemlock Lake includes a number of miles of riveted wrought-iron pipe which has been in use over 35 years and (2) because, partly on the strength of the success of the wrought-iron pipe, riveted steel was used for the second supply main made necessary in the early '90's. The Rochester water-works are also notable, among those of their size, on account of their now having 98% of the services metered.

Some old rotary piston water meters, in use since 1880 and 1881, recently showed a fair degree of accuracy after having been in use all that time. Remarkable results were also shown in tests of disk meters which had been in use for a long time.

The new Cobb's Hill Reservoir, put in use early in 1909, affords a storage capacity of nearly 150,000,000 gallons within the city limits, thus guarding against any possible accident in the long gravity supply mains. This reservoir is built on a hilltop and consists of a concrete wall backed by earth, together with a concrete bottom. At the base of the wall there is a gallery extending around the whole reservoir. Besides saving a cubic yard of concrete per linear foot of wall, this gallery affords an opportunity for leakage inspection. The leakage has been very small. The wall was built in blocks 20 ft. in length.

It may not be out of place to mention here a paper presented near the close of the convention, describing the purification plant of the Rochester & Ontario Water Co. This paper was read by Mr. James M. Caird, of Troy, N. Y. The company named supplies a small portion of Rochester and a number of suburbs. Water is drawn from Lake Ontario through an intake 4,000 ft. long, terminating in 40 ft. of water. The water is purified by passing it through horizontal filters under 235 lbs. pressure. Sulphate of alumina is used as a coagulant. This is sometimes supplemented by calcium hypochlorite, with a considerable saving in the alumina. Recently sulphate or alumina has been used at the rate of .7 gr. per gal., and calcium hypochlorite at the rate of .048 gr. per gal. When the turbidity of the lake water exceeds 10, a coagulating basin is brought into use. As many as 15 kinds of algae have been found in the lake water. When these are at their worst, copper sulphate has been used with good results.

During the evening devoted to Rochester, an interesting address was made by Mr. R. M. Searle, Vice-President of the Rochester Railway & Light Co. The burden of this address was the attempt of the company to cooperate with the city authorities, the other business interests and the people of Rochester in general in the advancement of the city, while at the same time endeavoring to give good lighting and street railway service. The company maintains a staff of some twenty technical graduates, devoted to advancing the manufacturing interests of the city by the economical use of power and light. Mr. Searle said that the members of this staff stand ready to look into the power and heating plant of any local establishment, and do not hesitate to advise the use of steam rather than gas or electricity for power or heating.

The Rochester evening concluded with a very interesting talk on the parks of Rochester by Mr. C. C. Laney, Superintendent of Parks. This talk was accompanied by a great number and variety of lantern slides. Particular mention may be made of the fact that one of the parks was donated to the city with the understanding that it would be devoted to an arboretum to be laid out under the direction of a competent landscape architect. On calling in the late Frederick Law Olmsted it was concluded that the area was not sufficient for the complete arboretum designed by the donor. Consequently, there was substituted a pinetum, and also a remarkably complete collection of shrubs.

Steel Pipe Lines.

Of a number of particularly valuable papers presented at the convention, we think none of the other authors will begrudge giving first place to the paper on steel pipe lines, by Mr. Emil Kuichling, Consulting Engineer, New York City. The impossibility of giving detailed information from this paper in a report of the proceedings of the whole convention will be understood when it is stated that Mr. Kuichling used about an hour in presenting an abstract only of the paper. All that can be attempted here will be to convey a general idea of the contents of the paper and the object of the author in presenting it. Speaking of the object first, it may be said that the author's aim was to make a thorough canvass of what is not known and what remains to be learned regarding steel pipe. On concluding his abstract of his paper, he said that an equally long paper could readily be prepared on the good points of steel pipe lines, but that pleasant task he would leave to others. In the course of his paper Mr. Kuichling gave a quite extensive review of the history and practice of steel pipe line construction. Naturally, the author devoted a considerable amount of time to the wrought-iron and the steel pipe lines at Rochester, with both of which Mr. Kuichling was connected. He also dwelt upon the relative merits of wrought-iron and steel plates as material for riveted pipe, and brought out clearly the evident great superiority of wrought-iron on the point of corrosion resistance. We hardly need remind our readers that comparatively few leaks have developed in the wrought-iron pipe line at Rochester after some 35 years of use, while in comparison the steel line developed many leaks within a few years.

The idea should not be gotten that Mr. Kuichling's paper was devoted wholly to the quality and corrosion resistance of steel pipe. He also discussed deformation. Here also his aim was to call attention to what is not known rather than what is known on the subject.

In discussing the paper, Mr. E. A. Fisher remarked that some people have an idea that the Rochester steel conduit is going to pieces. This is far from the truth. While there has been corrosion and leakage, the corrosion is confined to pitting, and the general strength of the conduit is not affected. Some $8,000 to $10,000 a year are spent in repairs consisting generally of uncovering the pipe each way from leaks and recoating it. Altogether, less than three miles have been recoated out of the total length of 26½ miles.

In the course of the discussion Mr. Francis H. Luce, of Woodhaven, N. Y., gave some information regarding the changes in the manufacture of enameled kitchen-ware which he thought explained why steel pipe was so much more readily attacked by rust than wrought-iron. Mr. Luce said that prior to 1890 the old-fashioned charcoal wrought-iron was used for kitchen utensils. Such utensils would last for a great many years. On changing to steel great difficulty was experienced in making the enamel adhere, and special means to that end had to be devised. He thought the lack of adhesion was largely due to the fact that the steel is not nearly so porous as the wrought-iron.

Mr. L. M. Hastings, City Engineer of Cambridge, Mass., stated that the steel pipe which he put in many years ago followed the Rochester specifications in general. The Sabin coating was used at Cambridge (as it also was used on the

sections of the Rochester pipe which have corroded least). Mr. Hastings stated that heating the metal before dipping the pipe to apply the Sabin coating opened the pores so that the coating penetrated better and thus had greater adhesion subsequently. So far as he knew, his pipe line was well preserved.

Mr. W. C. Hawley, of Wilkinsburg, Pa., who was formerly located at Atlantic City, told how the steel pipe laid across salt marshes had gone to pieces there. This pipe was laid against the advice of Mr. Hawley and Mr. Kuichling and against the wishes of the water commissioners, the city council setting itself up over all these and not permitting the use of wood stave pipe, as had been recommended by the engineers named. It is interesting to note that a 48-in. pipe line is now being laid, or will soon be laid, across the meadows at Atlantic City. The Atlantic City steel pipe line was laid some nine years ago. The pipe was coated with "mineral rubber," and had burlap wrapped around and pressed into the hot coating in order to afford protection in transportation. Pipe for the first half mile laid was carted across the meadows. Subsequently, the pipe was rolled 600 to 800 ft. across the meadows, which were frozen at the time. About a hundred leaks soon occurred in the portion of the pipe line, the pipe for which had been rolled, while only a few leaks occurred on that which had been hauled. This, of course, was explained by damage to the coating caused by rolling the pipe.

Mr. Allen Hazen expressed himself strongly in favor of steel pipe. He used it first at Albany a number of years ago. Recently at Springfield the lowest bids for conduit were at the rate of $2 per foot for steel pipe, against $3 for cast-iron pipe. He believed that ultimately steel would be used for all pipe lines larger than 36 ins. As to the use of wrought-iron, Mr. Hazen remarked that there was a difficulty in rolling this in large plates, especially where locked joints were used longitudinally. He understood that the ingot iron can be rolled in as large plates as the steel so that for this and other reasons he considered the ingot iron promising.

Mr. Morris Knowles stated that the considerable amount of steel pipe that had been used in and around Pittsburg had given general satisfaction. Mr. Robert Spurr Weston described the open-hearth steel process in its relation to the question under discussion. The large ingots from the furnaces cool in a way similar to the formation of ice in cakes; that is, with the concentration of the impurities at the center. When the ingots are rolled these impurities are unevenly distributed and thus give rise to unequal electric resistance in different parts of the plates, and consequently to concentration of corrosion since corrosion, as it is now known, is due to electric action. The first requisite of coating, of course, is to keep water from the pipe. The Sabin coating is better than others because it is baked on and in and thus excludes water. Unsatisfactory results sometimes had with Trinidad asphalt are in part due to the fact that this material includes some 25% of non-bituminous matter, which is dissolved by the action of the water, thus leaving the coating porous. The purely bituminous products are therefore more desirable.

Water-Works Income, Depreciation and Other Topics.

A paper on some studies of the growth of water-works income by districts having different classes of population and industry, as exemplified by the experience of the Indianapolis Water Co., was presented by Mr. F. C. Jordan, Secretary of that company. Yearly records, plotted as curves, showed great variations in income among the different districts studied. The curves also showed conclusively that in many cases the income was insufficient to meet fixed charges. The paper gave rise to considerable discussion regarding the proper basis for water rates and also regarding guarantees of revenue on extensions. Mr. W. C. Hawley stated that every private company was liable to do a losing business for years in some sections. His company requires a guarantee of 15% revenue on extensions. Mr. Leonard Metcalf discussed some of the broader aspects of water rates, including the attitude of public-service commissions. He thought that the paper by Mr. Jordan tended to show that the "going value" ideas of the Wisconsin Commission were wrong. Mr. James L. Tighe enlivened the proceedings by giving some very low figures of water rates and hydrant charges at Holyoke, Mass., and stating that notwithstanding this the water-works plant paid taxes just as it would if it was owned by a company instead of a city, and still came out even. Inquiries showed that, although the water department of Holyoke takes care of all capital charges, yet in these latter years of very low rates it has been paying interest on only a small remainder of the original bond issue.

An extended and valuable paper on "Depreciation in Water-Works and Methods of Accounting Therefor" was presented by Mr. Leonard Metcalf, Consulting Engineer, Boston, Mass. Mr. Metcalf divided depreciation into physical, functional and contingent, and discussed the advantages and disadvantages of the sinking-fund and the straight-line methods of computing depreciation. The sinking-fund method comports well with water-works experience and is in some respects advantageous to the consumers, but it involves more complicated bookkeeping than the straight-line method, or the plan of setting aside an equal amount each year for depreciation. In conclusion, Mr. Metcalf urged that water-works men should keep better records, particularly those bearing upon both long life and short life of the elements of their plants. The paper gave rise to considerable discussion. Mr. Kuichling called attention to the fact that many of the depreciation data, as well as large portions of the other data used by engineers, are ancient, having crept into the literature with no very good reason years ago and having been handed down over and over again since.

An interesting explanatory paper on the theory and practice of corrosion of metals was presented by Mr. Robert Spurr Weston, Sanitary Expert, Boston, Mass. This paper was illustrated by formulas based on the comparatively new theory of ions, and also by laboratory experiments. Mr. Weston remarked that tin-lined pipe is liable to corrosion because tin crystallizes and thus water can pass through the lining to the metal beneath. As a discussion of Mr. Weston's paper, Mr. Harold J. Stevens, of the Board of Water Supply of New York City, sent some remarks on the effect of hypochlorite of lime solution on metal pipes. Plain wrought-iron pipe was most readily affected, the inverts being eaten through in three or four months. Galvanized wrought-iron was partly filled by tubercles. Brass was less rapidly corroded than wrought-iron, and it would probably last two or three times as long before needing renewal or cleaning. Bronze float valves were in good condition after four or five months' immersion in the hypochlorite solution, but their seats were coated. Copper floats were eaten through in three months. Tin stood well. No observations were made on cast-iron mains as there is reason to believe that no damage to these need be expected on account of the small proportions of the hypochlorite which would go through the distributing system, and also because such hypochlorite as there was would be largely taken up by the organic matter. Wood stilling boxes have stood one and a half years' exposure to hypochlorite, but the wood was so soft that the edges of the boxes could be readily broken off.

The "Use of the Water Ejector for Transporting Sand" was reviewed by Mr. Morris Knowles, Chief Engineer, and Mr. John M. Rice, Division Engineer, Bureau of Filtration, Pittsburg, Pa., the paper being read in abstract by Mr. Knowles and accompanied by lantern slides. A written discussion of the paper, which was virtually a continuation of it, was sent in by Mr. Chester F. Drake, Superintendent of Filtration, Pittsburg. The chief object of Mr. Knowles' paper was to present the result of a series of experiments made at Pittsburg to determine the efficiency of sand ejectors and the best proportions of their various parts. This portion of the paper consisted largely of tables and diagrams. Mr. Knowles also presented a brief historical review of the use of the sand ejector. This began at London in 1886, when Mr. William Bryan, engineer of one of the water companies there, first employed it. This use was not for washing sand, as has been so common since, but for transporting it. A record distance of some 700 ft. of transportation was made thus early. The first use of the sand ejector in the United States was made at Lawrence, Mass., in 1895. This ejector was modeled by Mr. Allen Hazen after an ejector used on the water filters of Hamburg, Germany.

At Pittsburg during the first seven months or so of the present year, some 40,000 cu. yds. of filter sand were transported and washed at a cost of 12 cts. per cu. yd. This, of course, included cost of pumping the water used for the purpose Mr. Allen Hazen mentioned some extensive experiments on sand ejectors made by himself and Mr. F. F. Longley, formerly of the Washington filtration plant. Some valuable data have been accumulated which Mr. Hazen had hoped to publish before this time. He offered to incorporate some useful formulas based on the experiments in the discussion on Mr. Knowles' paper.

In the absence of the author, a paper on ice formation, by Prof. H. T. Barnes of McGill University, Montreal, was read by title.

The only other subject on the program remaining to be mentioned is "The Relation of Flies to the Transmission of Infectious Diseases," by Dr. Herbert D. Pease, Sanitary Expert, Board of Water Supply, New York City. Dr. Pease reviewed the important modern additions to the knowledge of this subject in an interesting and instructive manner, and then presented some very striking object-lessons to illustrate his points. These were in the shape of moving pictures, secured from abroad by Mr. Edward Hatch of New York City. The pictures gave the life history of the fly, and showed in a way that no one who saw the pictures will ever forget just how the fly infects food.

Convention of the Central States Water-Works Association.

Rather small attendance marked the convention of the Central States Water Works Association, held at Indianapolis, Ind., on Sept. 20-22, 1910. The papers presented covered a varied range, but were mostly of general character. The central feature of the program was the large amount of attention devoted to water purification and the problem of furnishing clean and bacterially safe water.

The State Sanitary Engineer of Kentucky, Mr. Paul Hansen, made a strong plea for fuller recognition by water-works managers of their duty to supply pure water. A few passages from the paper will illustrate:

> . . . We find otherwise competent and reliable water-works superintendents zealously defending water-supplies known to be polluted or subjected to the possibility of pollution. The argument used in nine cases out of ten is that the water cannot be harmful since no cases of typhoid fever or other diseases have been definitely traced to its use. This argument is sometimes used in the face of the presence of an abnormally high death-rate from typhoid fever. . . .
>
> It is unfortunate that the spur of competition, combined with a desire on the part of many municipalities and water companies to secure an installation at very minimum expense, has resulted in the erection of many filter plants unworthy of the name, which have done much to throw discredit upon the process of filtration in general. Added to this, there is often a lax and unintelligent operation of filter plants, under which conditions even the most carefully designed and constructed installation will fail to give acceptable results.

After a review of some principles which should guide in choosing sources of supply, or in deciding to adopt filtration, the author remarked as follows on hypochlorite disinfection:

> . . . there is still an opportunity for the water-works superintendent to furnish his consumers with a safe water at a trifling expense, both as to first cost of installation and subsequent operation, by the use of hypochlorite of sodium or calcium as a disinfectant. The writer is not prepared to state to what extent hypochlorite treatment may take the place of filtration, but it should be observed that disinfection does not remove any of the objectionable physical characteristics of the water; and moreover it is apt to impart a medicinal taste to the water which is almost certain to give rise to popular complaint.

A similar subject was treated by Mr. George C. Whipple, Consulting Engineer, New

York City, in a paper entitled "Clean Water as a Municipal Asset," Mr. Whipple reviewed the changing standards of purity as concerns drinking-water: earliest, the purely physical standards of clearness, and absence of color, taste and odor; next, the chemical standard, when everything went by nitrogen results; and to-day the bacteriological standard, when the colon bacillus is the criterion of pollution and the typhoid death-rate is taken as a measure of purity. While the change in standards is to be regarded as a rise of standards, the author deprecates the practice of going by bacterial results alone and disregarding the older criterion of physical cleanness. The hypochlorite disinfection process is apt to be credited too highly, for, Mr. Whipple says:

> It does not improve the appearance of water, nor its taste, nor odor—on the contrary its tendency is to add to the odor of some waters, especially if used in too large doses. It does not make a water visibly clean, it kills bacteria. Such being the case its scope of usefulness is obviously limited. Those in charge of water-supplies where the water is always clean, or of supplies that need an emergency treatment to destroy a sudden infection, may find in this agent a safe, effective and inexpensive germicide. As an adjunct to filtration the use of calcium hypochlorite in the future is likely to be large, as it offers an additional safeguard and supplements the work of the sedimentation basin and the sand bed. It sometimes enables higher rates of filtration with less coagulant to be used, and thus reduces cost. There is danger, however, that this substance may be regarded as a means of purification complete in itself, that it may be employed with uncleaned waters without other means of purification, and that because of its cheapness it may find adoption in situations for which it is not appropriate. There is danger, also, lest water-supplies suffer from its faulty application, that is, through unsatisfactory apparatus and neglect. Already several unfortunate instances of this might be brought to your attention. In short, through zeal to raise the bacteriological standard of water, coupled with motives of economy, there is danger lest the virtue of cleanliness be overlooked.

The remainder of the paper gave a summarized statement of the different kinds of purification required, and emphasized specially that filtration is becoming more and more necessary for even the best waters, as those from the Great Lakes. Approximate figures of cost and value of purification were cited from the author's book "The Value of Pure Water," the value ranging from $5 to $200 and the cost not over $10 to $15 per million gallons. As to softening hard water, the balancing of cost against value to the community does not in every case make it economical, whereas purification always shows a net saving.

There was little or no discussion on the preceding two papers, but "The Acid Waters of Western Pennsylvania," by Mr. C. E. Trax, of McKeesport, Pa., brought out some remarks. This paper, of which we hope to give an abstract in our next issue, described the condition of the Youghiogheny River and gave figures as to its acid percentage and the variations thereof. McKeesport is treating its water to remove acidity and hardness. The author charged coal-mine washings with being the source of the acidity, and seemed to be pessimistic as to the prospects for future prevention of this pollution. Mr. J. B. Ayres, of the Board of Water & Light Commissioners of McKeesport (and also connected with the National Tube Co.), said that the water of the Monongahela River is used over 22 times by the 65 steel mills between Brownell and Pittsburg. As to whether the pollution should be prevented or the water should be purified before using, he suggested that the burden should be placed where it could most easily be borne. Dr. J. N. Hurty, State Health Commissioner of Indiana, dwelt on the extent and variety of stream pollution by industrial operations, and on the fact that complaints against industrial plants were often exaggerated. He thought that it might prove easier to treat the acid water after contamination than to attempt to keep the mines from polluting the water.

"Repairing a 16-In. Main" was read by Mr. C. W. Wiles, of Delaware, O. Difficulties in applying a sleeve over a crack in a main were described.

Tests of pipe under air pressure and under hydraulic pressure were described by Mr. W. R. Conard, of Burlington, N. J., under the title "Some Experiences in Tests of Pipe." Pipe furnished for gas mains which were desired to be absolutely tight was specified to meet not only a hydraulic pressure test but also a test of 50-lb. air pressure, soap suds being applied outside to show leaks. Out of 7,691 lengths tested by both methods, 41 leaked in the hydraulic and 458 in the air test; the percentages are 0.53% for hydraulic and 6.0% for pneumatic testing. In view of the comparative figures the author suggests that the pneumatic test is valuable for water pipes also. In discussion, however, some members expressed the view that the minute leaks revealed by the pneumatic test would be sealed by the pipe coating or by rust.

Under the title "Floods, Conservation and Relation to Water-Supply," Mr. Morris Knowles, Pittsburg, Pa., made a plea for general interest in and sympathy with the headwater control studies which were started in Pittsburg two years ago with a view to developing means of protecting Pittsburg from floods. The threatened shut down of the Brilliant pumping station in 1907, and the water-supply troubles which arise in periods of drouth, were used to enforce the plea that water-works men should assist Pittsburg's efforts.

The West Washington Street pumping station of the Indianapolis Water Co. contains steam-turbine centrifugal pumps, which were seen by the Association during an inspection trip on Sept. 21. Mr. C. H. Hurd, Chief Engineer of the company, based an instructive cost-comparison paper on the performance of these pumps, which was presented under the title "The Steam Turbine Centrifugal Pump." Taking as example a plant pumping ten million gallons per day on the average, with two units of this capacity, working against a head of 200 ft.; with coal at $2.50 per ton; with depreciation and interest at 9%; assuming a high-duty reciprocating pump equipment to cost $180,000 and to show a duty of 150,000,000 ft.-lbs. per 1,000 lbs. steam, and a centrifugal pump equipment to cost $90,000 and to show a duty of 80,000,000; and with boilers evaporating 6 lbs. water per lb. coal; he deduced that the centrifugal pump plant would show a saving of $800 annually. If coal cost $2.80 per ton the total annual costs would be equal; with higher-priced coal the high-duty plant would be more economical, with lower-priced coal the centrifugal plant.

A short paper by Mr. C. E. Inman, Warren, O., discussed the subject "The Business End of a Water Company and Its Dangers."

Coal economy was discussed by Mr. F. W. McNamee, of Wabash, Ind., in an impromptu address. The Wabash pumping station by introducing a bonus system for its firemen, based on the percentage of carbon dioxide in the flue gases, has achieved a marked reduction of coal consumption. The gases from the flue of each boiler are sampled by a continuous sampling bottle, arranged so that its water contents of 5 gallons drip out in 12 hrs. All bottles are tested the following day, and the fireman is paid a bonus on this test. For 6% CO_2 he gets 15 cts. bonus, for 14% 70 cts. bonus.

The Association elected as principal officers for the coming year: President, A. L. Holmes of Grand Rapids, Mich.; Vice-President, W. J. Scroggins of Wheeling, W. Va.; Secretary, W. A. Veach of Newark, O; Treasurer, A. W. Inman of Massillon, O.

The 1911 convention will be held at Cleveland, O., by vote of the meeting.

A Temporary Hypochlorite Plant for Treating the Water-Supply of Milwaukee, Wis.

By CHARLES J. POETSCH,* M. Am. Soc. C. E.

On June 18, 1910, the new Health Commissioner of Milwaukee reported that the number of typhoid cases in this city were on the increase and blamed the water-supply as the cause. In a conference with the mayor, the health commissioner and the city engineer, it was decided to sterilize the water immediately. On June 21, a very temporary plant, consisting of a single tank in which a hypochlorite solution was mixed, was set up over the water-supply tunnel. The solution was fed to the supply before it reached the pumps by means of a 1-in. pipe, the flow being regulated by a valve. One week later the present plant was started in operation.

The plant now consists of a mixing tank 3 ft. in diameter and 4 ft. high and of two solution tanks, each 8 ft. in diameter and 7 ft. high. The mixing tank has a stirring device operated by hand. The tanks are of cypress lumber, set up outside the pumping station over the supply tunnel and connected with galvanized-iron pipes with the supply. The flow is regulated by means of ordinary valves, according to the amount of pumpage in the station. Six pounds of hypochlorite of lime are used per million gallons—which seems to be ample according to a recent test made by the city chemist. Daily tests of the raw water and of the treated water are now being

Temporary Hypochlorite Plant for Treating the Water-Supply of Milwaukee, Wis.

made by the health department. The results of these tests show about 200 bacteria per c. c. in the raw water and from 2 to 6 in the treated water.

Should it be necessary to continue the treatment of the water, it is proposed to construct concrete tanks, close to the pumping station, with the mixing tank inside of the building, so as to be able to do the mixing by mechanical means; the solution could then be fed into the pump wells. Wooden tanks are liable to be digested by the hypochlorite in the course of time.

The pumpage is now 50,000,000 gals. per day. The cost of treating this quantity is $5.25 for labor and $4.95 for chemicals, making a total of $10.20 per day, or 20 cts. per 1,000,000 gals. The installation cost was about $225.

[The water-supply of Milwaukee is drawn from Lake Michigan at a point some 8,000 ft. from the shore and at a depth of about 50 ft. See Eng. News, June 30, 1910, p. 763, for some notes on the recent typhoid outbreak and the possible sewage pollution of the supply; also Sept. 19, 1895, for detailed description of the water-works tunnel. Judging from information which has come to us from various sources there appears to be reason to doubt whether the water-supply was responsible for the outbreak, but the data which we have seen have been meager.—Ed.]

*City Engineer, Milwaukee, Wis.

THE MUNICIPAL TRAMWAY SYSTEM of Sheffield, England, is reported on by Consul C. M. Daniels in the Daily Consular Reports of the U. S. Bureau of Commerce and Labor, Sept. 12. A profit of $191,020 for the year ending March 31, 1910—an increase of $17,256 over the previous year—was shown. The total capital expenditure now amounts to $4,882,700, the increase in the past year having been about $208,000. The annual income for twelve months was $1,455,590 and the total operating expense was $883,348, leaving a gross profit of $572,242. The difference between this and the net profit represents interest on stock, mortgages, improvements and contributions to the sinking fund. The average operating cost is given as 11.68 cts. per car mile, compared with 11.77 for the previous year. The scale of fares shows that the average distance passengers travel are half a mile for 1 ct., 2½ miles for 2 cts., 3 miles for 3 cts. and 5 miles for 5 cts. On July 15, 1910, the management reduced the rates of fare by raising the age limit of half fare to twelve years. The average receipt per passenger mile is 0.386 ct.

Separately-Molded Sections for a Concrete Sea Wall.

In the construction of new sea walls at Sydney, N. S. W., for the Sydney Harbor Trust, Mr. W. E. Adams, A. M. Inst. C. E., has in two separate locations designed and built a new type of concrete retaining wall, in which L-shaped sections are separately molded on shore and swung into place. These walls are described in the August, 1910, issue of "Concrete and Constructional Engineering" of London, and from that paper we have abstracted the following description.

Fig. 1 shows the general position of the first wall built. It will be noted that a dock landing is made on a sloping foreshore by dumping in stone to form a sloping underwater face with a horizontal surface at the water level but some 14 ft. below the dock level. Just inside the top of the slope, on the level surface, is placed an L-shaped retaining wall holding back an earth fill to the level of the original slope and extending seaward from this wall an ordinary timber and pile quay is built. The particular wharf here shown is 490 ft. long and in calling for bids for the construction of the retaining wall alternate designs of a separately-molded concrete wall, a solid concrete wall and a stone wall were asked for. The first of these proved the cheapest and was then built.

Fig. 2 shows the general design of the wall. It consists of separate L-shaped counterforts 12

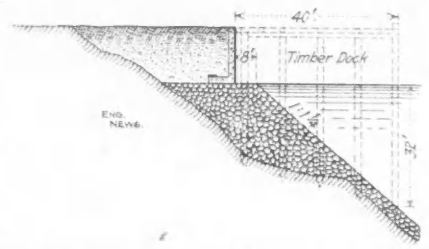

Fig. 1. Section Through Quay and Wall, Miller's Point, Sydney, N. S. W.

ins. thick, 13 ft. high and 8 ft. deep, with a solid horizontal base and a trussed vertical member. These are spaced 6 ft., c. to c., and act as counterforts to hold the front slabs which retain the earth. The bottom of each counterfort has a wide slab to spread the pressure and another slab is placed on top of the far end of the horizontal of each L to guarantee a larger distribution of the earth load which holds the walls against overturning. All of these members are reinforced, although the size and disposition of the metal are not shown in the original drawings.

In the construction, the rock bottom was first rammed flat with a large wooden templet swung to a pile-driver, and then the front foundation plate was placed and also rammed into position. Bagged concrete was then placed back of each foundation plate to make the bearing of the counterfort level, all of this work being done above water at low tide. The counterforts, which had been molded in a vertical position on shore some time previous, were then picked up by a derrick and swung into place, also at low water when the upper part of the footings were in air. Finaly, the slabs were swung against the projections in the counterforts and the fill made from shore as the earth could be procured.

All of the walls in the vicinity of the one described sank after being placed, as the ground thereabouts is quite unstable, and the concrete wall proved no exception to the rule. During the progress of the filling the whole wall sank bodily from 5 to 7 ins., and during the first six months after the completion of the work there was an additional settlement of from 1 to 2½ ins., but after that no further movement was observed in the three years since the completion of the wall. The settlement was uniform and no cracks have appeared, which was not the case with adjacent stone walls.

In addition to the ordinary earth pressure, a live load of 200 lbs. per sq. ft. was assumed as a surcharge above the wall. With this heavy load the maximum pressure on the foundation is only 2 tons per sq. ft.

This wall proving very successful, a second wall of much greater size but somewhat different design was built. In the second wall, the disposition of the stone filling and the timber wharf was the same as in the first, but the wall had to hold 21 ft. 6 ins. of fill with an allowable maximum footing pressure of but 2 tons per sq. ft. The wall shown in Fig. 3 was designed. This is also of reinforced concrete, but differs from the first wall built, in the absence of any extra bearing plates or intermediate vertical slabs. Each counterfort is 3 ft. 6 ins. wide in front, 21 ft. 6 ins. high and 15 ft. deep with solid sections about 12 ins. thick. Each one

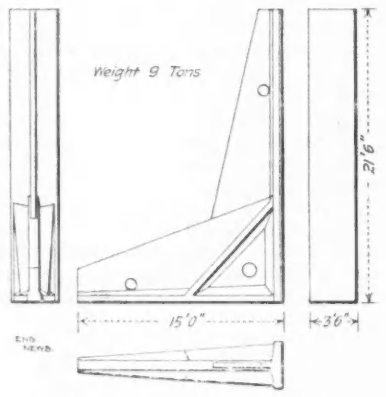

Fig. 2. Detail of Separately Molded Concrete Wall Placed in Location, Shown in Fig. 1.

weighs about 9 tons, while the counterforts described above did not exceed 3 tons.

The footings in the second wall were under water at all times and were prepared by pulling over the stone heavy straight edges guided by divers. In placing the sections a heavy waling piece was fastened to the nearest line of piles, and the counterforts swung in place from chains attached to logs driven through the three holes shown in the drawing (Fig. 3). Each member was swung in snug against the adjacent one just placed and when the whole line was com-

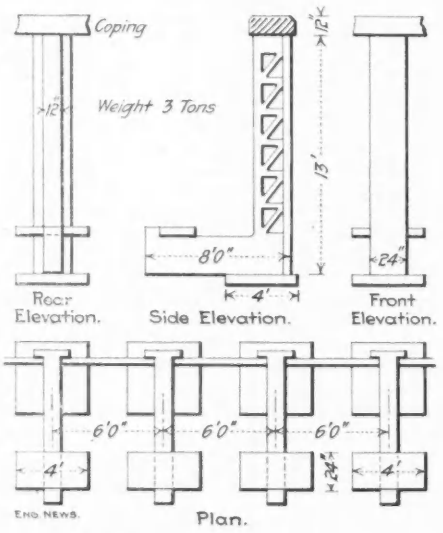

Fig. 3. Detail of Large Separately Molded Wall Placed in the Deep-Water Portion of Sydney Harbor.

pleted the filling was commenced. Here, as in the first wall, a settlement and lateral movement occurred, but in neither direction large enough to cause any trouble. The wall has been in satisfactory service for about 18 months.

Messrs. Gummow, Forrest & Co., Ltd., were the contractors on both walls. The contract price of the first was £3 per lin. ft. and of the second £8 15 s. per lin. ft. It was estimated that a gravity wall for the second location would cost £13 per ft.

Test of a 6,000,000-Gal. Pumping Engine at Muskegon, Mich.

The ability to reach a duty figure, with comparatively small pumping engines, that would do credit to those of the largest type has been several times reported in Engineering News. In our issue of Aug. 18, we noted that two 5,000,000-gal. horizontal, triple-expansion, fly-wheel engines installed at the Canarsie Pumping Station, Brooklyn, N. Y. City, showed duties of 151,474,000 and 156,481,000 ft. lbs. per 1,000 lbs. of steam, respectively, under somewhat adverse running conditions.

A recent similar instance has been drawn to our notice, that of a 6,000,000-gal. triple-expansion, single-acting, self-contained engine, built for the municipal water-works of Muskegon, Mich. The auxiliaries are attached to the engine and the whole unit exclusive of suction and discharge piping weighs 225 tons. The valve gear is a Corliss type driven directly from the main shaft; an automatic speed-pressure governor regulates engine speed and water pressure; a surface condenser is placed in the suction line and its single-acting air pump, the feed pump and a compressor to supply air to the air chambers are all directly connected to the low-pressure plunger head.

The requirements laid down for this pumping engine were for a capacity of 6,000,000 gals. per 24 hrs. against a domestic main pressure of 70 lbs., not less than 135,000,000 ft. lbs. per 1,000 lbs. of steam at 125 lbs. boiler gage pressure. A pressure of 125 lbs. per sq. in., was demanded on the water mains for fire service.

The official tests were made by H. A. Allen & Co., of Chicago, Consulting Engineers. On account of unfavorable conditions it was not possible to conduct the test under contract conditions. The boilers were judged unable to stand safely a pressure of over 90 lbs. gage. The quantity of water pumped was determined from plunger displacement corrected for slip (0.219% by test). Moisture in the steam was found by a throttling calorimeter and readings of all instruments, gages, thermometers were taken every ten minutes with the exception of indicator cards which were taken once an hour.

The test showed a capacity of 6,589,200 gals. per 24 hrs., under 70 lbs. water pressure and a duty of 141,536,000 ft. lbs. per 1,000 lbs. of steam at 90 lbs. gage pressure. The duty corrected for contract steam pressure of 125 lbs. gage was 151,695,000. The change from low to high speed for fire service was made in from 15 to 20 seconds.

New boilers are being installed at the Muskegon station, so that steam can be obtained at 125 lbs. gage. When this work is completed it is expected that tests will be made showing improvement over the computed duty of 151,695.000 ft. lbs. as the corrections for this figure were considered very conservative.

THE PROFIT-SHARING SCHEME of the British Columbia Electric Railway Co. in Victoria, Vancouver, and New Westminster, was revoked on June 30, 1910, according to a report by Consul A. E. Smith, of Victoria. This system, which was put into service July 1, 1903, gave a share of the profits to all employees who had been one year in the company's service: First, a dividend of 6% on the capital of the company was declared, and then the balance of the year's profits was divided into three equal parts, of which two went to the stockholders and one to the employees. This last sum was equally apportioned among all the employees irrespective of their rank or salary. The annual bonus amounted during six of the seven years of operation to $30, $35, $40, $45, $62 and $57 per employee respectively. The figure for 1907 was not reported. It is stated that the men thought their share of the profits insufficient and their discontent was shown in demands for readjustments of shares and wages. The maximum figure paid conductors and motormen at that time was 31.5 cts. per hour, and the minimum was 20 cts. The men worked on an average nine hours each week-day and about eight hours on Sunday. The company, at the request of its employees, on the date mentioned abandoned this profit-sharing scheme as a failure and averted a strike by adopting a new wage schedule for motormen and conductors varying from 22 to 35 cts. an hour. The employees have now formed a mutual benefit association to which each man contributes 50 cts. a month and is allowed $1 per day during sickness. The company employs about one thousand men.

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The political situation, as it has developed up to now, shows a bright and gratifying feature, namely the total absence of any reference to the Panama Canal construction. This is final proof that the Canal is now safely and finally out of politics, as far as its construction is concerned. The loud-voiced "bet you ten to one" objectors of not so long ago, men who wanted to stop the work, uproot the engineering organization, revolutionize the plan, introduce iconoclastic methods, and the like, are sitting quietly on the rear seats or else have gone out of the meeting-hall or have moved out of town. Meanwhile at Panama the digging and damming go forward on schedule, quietly, orderly and rapidly.

The movement toward universal public ownership of water-works received a set-back at Indianapolis last week. The mayor of that city, Mr. Samuel Lewis Shank, welcomed the Central States Water Works Association with some well-chosen remarks. A report of the speech contains the following notes:

> The Mayor complimented the management of the Indianapolis Water Co. . . . .
> He believed that water-works could be more economically administered by a private company, and declared that the Indianapolis Water Co. in emancipating the people from the old well system had conferred a great benefit on the public.
> . . . . he confessed that he had never received any water bills under Mr. Boyd's able management. [The president of the company is L. C. Boyd.—Ed.]

The address is referred to in the report as being "of a semi-humorous character," and it was excellently received, eliciting "repeated rounds of applause." It is human to applaud a more fortunate fellow-being, and surely one who receives no water bills is more or less fortunate.

Failures of wooden trestles are rather more common than should reasonably be expected. Being under closer supervision than other parts of the roadbed it might be assumed that trestles could not often get into a dangerous state. But a fairly regular succession of reports of trestle-bridge failures indicates either that the supervision is not close enough or else that the results are bound to be uncertain. The latter is by no means improbable. Where the members of a composite structure all deteriorate progressively but at different rates, it is not easy to renew each part at its proper moment; and unfortunately any part may cause a failure: pile, sill, post, cap, stringer or tie. Renewal of the structure as a whole is apt to be timed to the average condition rather than to the condition (or probable condition) of the poorest part. Something is to be said in favor of placing a large margin of safety on life of timber trestles, and making complete renewals at intervals so determined, irrespective of condition.

Occasional washouts of trestles crossing small creeks or dry runs also direct attention to trestle piling. Every year several failures of this class occur, and though not all produce a train wreck, yet several of the worst wrecks of the last decade were due to such failures. It is fair to conclude that in the territory from the Missouri River to the Coast a dry run should always be regarded as a bank-full torrent, in planning structures. Since these ravines usually have a steep slope, they offer the possibility of great scouring power, and foundations of the most resistant type should be provided.

Even if Engineering News were devoted to the industry of exploiting the reader by hiring out its reading pages, a modest request just received would probably slip in gratis. The request is worded in business-like fashion, quite concealing the transcendent importance of the subject-matter offered for publication:

> Engineering News, New York.
> Gentlemen:
> Please insert this brief article in your next publication and send bill to me.
>
> Yours truly,
>
> ———— ———— ————.
>
> Wed., Sept. 14, 1910.

We omit the name and address in deference to the sender's wish and because we do not want the secret suggested in the "brief article" to get away. The sheet enclosed bore this message:

> It has been discovered that engines are not needed in vessels, automobiles, flying machines or locomotives. The blades on the turbines should move in opposite directions.
>
> Address
> Engineering News.

The last three words are unauthorized, for we decline to forward replies unless the secret is first revealed to us. But isn't the discovery a thing of wonderful promise? There is just a chance that it is partly anticipated by one of the New England crop of flying-machines, which, according to a veracious contemporary, not only dispenses with a motor (to save useless weight), but also is equipped with a cellar, a high-stoop front, and a back stairway for the "help."

Data on the discharge of overflow dams, like those presented by Mr. Martin on another page, are needed in much greater fullness than yet available. Hydrographic measurement is not the only field of use for such data. Dams and other "broad-crested weirs" occur ever more frequently under conditions where an exact knowledge of the relation between discharge and gage-height is important. Nearly every hydraulic power plant gives occasion to keep a continuous record of water consumed and water wasted, in the shape of a record of gage-heights on a dam. The question of what crest depth will be required to pass a given flood-discharge, important though it be, is of second importance only. The chief need is to collect data and establish formulas which will make the overflow dam a satisfactory metering device.

What success can be obtained in this effort is not yet evident. The notably different exponents deduced by Mr. Martin suggest that perhaps a formula will always be of uncertain value for any dam other than the one which established it. The difference between the equations found for the Austin and La Grange dams, moreover, gives the warning that general similarity in form will not ensure similar discharge constants. But as the Austin dam was metered only at low heads the latter possibility is not fairly proven. In fact every question relating to discharge formulas for dams is yet open. The only obvious thing is that it is important to use special care, in building overfall dams, to secure a straight, level crest-profile.

It will be desirable to determine, if possible, one or more shapes of crest which will each give an unchanging discharge-formula at all heads, for all heights of overfall, and for any required top width. A few such shapes, provided also they are accurately reproducible, would meet all needs in the design of dams. For, the idea that overflow dams must be shaped to the flowing sheet of water is no longer of great currency, we take it. But before such standard crest sections can be fixed, it is necessary to have a large accumulation for discharge data and formulas for large dams.

Compulsory legislation requiring the use of block signaling on electric interurban railways may be looked for by the railway companies if they do not show a much greater disposition to take the initiative in this much needed reform. The numerous accidents under the train dispatching system are undoubtedly giving rise to an antagonism against these railways on the part of the public (somewhat similar to the earlier "granger movement" against steam railways), and this may crystallize in a powerful demand for legislation to remedy the conditions as they now exist. In this case the history of operating methods on steam railways is repeating itself on electric railways. With a frequent service of trains, the operation of local and express trains, and the running of trains at speeds of 50 to 65 miles an hour (and all this usually on single-track lines with passing places), there is small reason for continued dependence upon the train-dispatching system, whatever precautionary methods may be attached to it. The deficiencies and dangers of this system as developed in steam railway operation are becoming more and more evident on electric railways, due to the increase in traffic and in speed. With light traffic and good discipline the system is fairly satisfactory, but certainly it is not adapted for the safe and efficient handling of dense traffic and high-speed service. This has been pointed out many times, and has been proved by its innumerable failures resulting in accidents. The recent very serious collision of an express and local train on an Indiana electric line is but the latest of these cases.

It is pretty well recognized and admitted that the block-signal system is the only safe method of handling such traffic, although, unfortunately, it is not so generally recognized that it is a means of facilitating traffic as well as safeguarding it. The block-signal system has been adopted very extensively by steam railways (partly under pressure), but only to a very limited extent by electric railways. One reason for this is that the latter are less able to incur the initial outlay for such equipment as is used by the former, while comparatively little has been done towards developing less elaborate and costly systems adapted to the operating and financial conditions of electric railways. It is probable also that the railways themselves (following out the history of signaling on steam railways) have little desire for such systems, and would rather oppose than encourage their introduction. Nevertheless, it appears to us that here is a wide field for the signal engineer and the signal manufacturer, a field which is opening slowly, but which may be opened more rapidly (and perhaps arbitrarily) in the future by the aid of legislative enactments. It should not be difficult to devise a simple automatic block system adapted to meet the particular conditions and at a reasonable cost.

As a general thing the discipline on lines of this class is less efficient than on steam railways and it has been objected that the men will be likely to disregard signals. Even assuming that the discipline remains unimproved, there is a good answer to this objection in the fact that on electric railways the signals are largely within view of the passengers. The front of the car is usually open, and passengers can see ahead of the train, which is not the case on steam railways.

Disregard of signals may then become a matter of public knowledge and eventually of specific complaint. Furthermore, it should be a simple matter to equip a car with an audible signal which will notify passengers of the action of a motorman in passing a "stop" signal. The railways and their official organizations might well give heed to the shadow of coming events and prepare to operate their traffic on a safer and more efficient system than that now in general use.

Projected Railway Construction in New South Wales.

An extensive program of railway construction and improvement is projected by the Government of New South Wales, and is one of the issues on which the public will pass in the pending election. It appears that the traffic has outgrown the capacity of the present railway system, particularly at the season when the crops are to be moved. It is proposed to double-track the main lines on which traffic is heaviest, to enlarge the terminal facilities at Sydney and to build extensions into districts in which the settlers are now without railway facilities. The total expenditure involved is estimated as at least $20,000,000 and probably a much larger sum.

In the London "Times" of Aug. 27 an Australian correspondent discusses the situation and urges that the railway system of New South Wales has been developed on wrong lines.

As explained in the paper by Mr. Jas. A. Smith, published in our issue of July 14, the railway lines of New South Wales all "fan out" from the seaport city of Sydney, the principal city of the colony, which contains a large proportion of the colony's population and wealth. Thus Sydney is the sole market and export point for the colony's products, while the city has by far the best natural harbor along the entire 600 miles of coast of New South Wales. Moreover, all the expenditure on improvements and terminal facilities has been concentrated at this point, and the two or three other possible sites for commercial harbors on the coast have been almost wholly neglected.

Besides this, according to the writer quoted, Sydney is surrounded by a broad area of barren, mountainous country which has to be surmounted by all the railways which radiate from Sydney. Further, many fertile sections of the colony have to submit to a haul of 400 to 600 miles on their products to the Sydney market, when a haul of only half or a third the distance over favorable grades would take them to another seaport were that seaport developed and railway lines planned and built to take traffic in that direction. The correspondent urges, therefore, that English financiers should be wary of supporting the New South Wales proposal for enlargement of the present system unless the colonial government will agree to undertake the development of ports to compete with Sydney and railway lines to supply them with traffic.

The question at issue is one that is arising with increasing frequency not only in Australia but in many other countries; and it is one by no means easy of solution. It is doubtless true that Sydney has used its influence to maintain its position as the only market and prevent the improvement of rival ports and the construction of railways leading thereto; but it does not follow necessarily that this policy is not on the whole for the benefit of the colony, if its railway system were properly developed and operated. It is proverbially a difficult thing to change existing currents of trade, and this is particularly the case where it is attempted to build up a new shipping port to compete with long-established commercial routes.

The increasing size and draft of ocean carriers has operated in favor of the large ports and to the detriment of their smaller rivals. The owner of a 10,000-ton freight vessel will not send his ship to a small port where variations in the cargo market are great and where perhaps there is risk of accident in the entrance channel. Again, the investment necessary to develop a port and supply it with such modern facilities that it may do business economically is vastly more now than a generation ago. Thus it has happened in the evolution of commerce in the United States, for example, that the bulk of the export business has been concentrated at a few great ports: Boston, New York, Philadelphia, and the cities at the mouth of Chesapeake Bay on the North Atlantic coast, and in the South, Savannah, the St. John's River, New Orleans and Galveston.

No one questions nowadays the advantage of such a number of ports as this. Even candid New Yorkers recognize that that city could by no means expect to maintain the position it once held as the chief outlet to Europe for the grain from Western farms.

The great advantage to the producers of the Southwest of the development of the port of Galveston is unquestioned. It does not at all follow, however, that continued multiplication of commercial ports would be economically desirable. Galveston's success does not mean that half a dozen other possible sites for ports on the Texas coast could repeat it, nor would it be economically desirable that the money should be invested to develop that number of ports if two or three can easily handle all the business.

It may be pointed out, too, that the question whether or not the development of a new port will be economically justifiable turns upon the ability of the railways to move freight over the road at low cost. We showed some months ago (Mar. 3, 1910) in these columns that under American methods the cost of hauling freight had been reduced so low that on a large proportion of traffic the cost of handling at the terminals is far greater than the cost of hauling between them.

We make no pretence of passing judgment on the problem presented in New South Wales. What we would point out is that no safe conclusions can be based on statements that the haul to Sydney is two or three hundred miles greater than the haul would be to some shipping port as yet undeveloped over railways not yet built.

Duplicating transportation systems where an existing one is already able to move all the traffic has been well demonstrated to be against the public interest, even where the business is carried on by private corporations and an element of competition enters in. In Australia, where the railways are owned by the Government, there is even more need of caution in the use of the public credit to establish ports and railways duplicating, in a certain degree, those now existing.

Some Water-Works Questions of the Day.

The programs of conventions of technical societies often give at least a general idea of what questions are most perplexing at the time to the membership of the associations. It will, perhaps, be interesting to examine the programs of the three water-works conventions held this year from this point of view. Two of these conventions took place during the past week, the New England at Syracuse and the Central States at Indianapolis. The third was held last April in New Orleans. At the New England and Central States conventions the papers were few in number compared with the very lengthy list submitted by the American Association at New Orleans. In fact, the number of papers at New Orleans was so large and the subjects so diverse that the program alone does not throw much light upon the question of what topics were uppermost in importance in the minds of the members. One must therefore turn to the discussions of the several papers in order to determine which subjects aroused the greatest interest. Unfortunately, with so very lengthy a program as was presented at New Orleans, it often happens that after a paper has been presented no one ventures to discuss it, either because of weariness over the long list of papers presented and yet in sight before the session, or else because it is felt that any attempted discussion would make it impossible to carry through the program. Nevertheless, any one attending the New Orleans convention would form a fair idea of the relative importance of the various papers from the way in which they were received.

One thing that must be considered in attempting to use a convention program as a means of determining what subjects are giving rise to the most thought on the part of members of the association is the way in which the programs themselves are prepared. We need hardly say that there are various ways of getting up a program for a convention. Some societies leave the matter to their secretary and others have a program committee. Then, too, quite regardless of whether the program is in charge of an individual or a committee, there are very different methods of determining what subjects shall be brought before a convention. Some program makers send out a blanket request to a considerable number of members asking them individually to submit papers and leaving to them the choice of subjects; other program builders ransack their brains to find a list of topics which in their opinion will add to the interest and variety of the convention program and then try to fit authors to the subjects. Too often the chief effort is to get a large number of papers, quite regardless of the timeliness of the subjects or the experience and ability of the authors. Rarely are the subject and author fitly joined and the program so limited in extent and the subjects so selected as to give time and inspiration for discussion—and a full and free expression of opinion from a goodly number of representative men is generally worth ten to a hundred times more than the papers themselves.

Elsewhere in this issue will be found reports of what took place at the meetings of both the New England and Central States Water Works Associations last week. An account of the last meeting of the American Water Works Association may be found in our issue of May 5, 1910. Leaving to such of our readers as are interested the drawing of conclusions as to the methods employed in making up the programs of these three conventions, we will proceed to a few remarks on what appear to us to have been the leading topics presented at the three meetings. In making our choice we shall use our own judgment to some extent as to the relative importance of the subjects discussed; that is to say, we shall take into consideration not only the prominence given to the subjects at one or more of the conventions but also the importance of these subjects in and of themselves.

Water purification seems to deserve first place. It is significant to note, however, that for the most part the subject was not considered so much in its broad, general aspects as in some of its special phases, such as softening, sedimentation, removal of iron, and particularly the use of hypochlorites for bacterial reduction.

The use of copper sulphate for the prevention of trouble from algae received scarcely more than passing mention. This aid to the water-works man has found its place, and for the time, at least, needs no discussion.

Ozone as a means of water purification does not appear to be deemed worthy of much consideration by water-works men in 1910. This is almost as true outside as within convention halls, for although ozone is being given a trial at a few works almost every one seems to regard it as being in a rather unsatisfactory experimental stage of development.

Next to water purification in importance it seems quite safe to place depreciation. It is true that this subject is giving rise to more concern among those having to do with private water companies than among those whose interests are chiefly in the municipally-owned plants, but even among the latter the subject is being given a degree of attention far in excess of anything thought of up to four or five years ago. The question is looming up in water-works valuation proceedings upon every hand and also in connection with the everyday operation and financial management of water-works, both private and public.

Closely allied to depreciation but also of importance in nearly the whole field of water-works operation is the subject of water-works account-

ing. Obviously, this subject is of too special and complicated a nature to be wholly within the grasp of the water-works superintendent, but this class of officials is beginning to realize the importance of a proper system of accounting and the desirability of uniform accounts and statistics. While but a relatively few individuals connected with the various water-works associations are taking a direct hand in the movement for uniform water-works accounting, yet it is believed that a large number of water-works superintendents and engineers are deeply interested in the subject and welcome the careful attention which is being given to it by the U. S. Census Bureau and by a few members of the several water-works associations.

Water rates are almost sure to come up in one form or another at any gathering of water-works men. For many years very little scientific consideration was given to the subject. Papers presented at the conventions consisted chiefly of tabulations of rates charged in various cities. Slowly the idea is gaining ground that the rates charged in one city have very little bearing upon the justness or unjustness of the rates charged in another. Very little progress, however, has yet been made among the water-works fraternity at large towards putting the framing of water rates upon a just and reasonable basis. This remark applies more particularly to flat or schedule rates than to meter rates. It is doubtful whether schedule rates will ever be put on a scientific basis, owing to the many complications which they involve and to the fact that any effort to make scientific schedule rates is liable to be upset by the almost criminal waste of water indulged in by many consumers unless they are compelled by the use of meters to pay for all the water which passes their faucets.

Where all, or practically all, the services are metered, the framing of just and reasonable water rates is a comparatively simple matter, provided due consideration is given to the division of the cost of water service between the private consumers and the general public—meaning by the latter the city itself through its use of water for fire protection, street cleaning, sewer flushing, parks, schools and other public buildings.

Associated closely with the question of water rates is the matter of extensions of distributing mains and whether or not these shall be made only on guarantees of sufficient revenue from abutting property owners or householders to pay at least fixed charges while the connections are few. An alternative is the assessment of the whole or part of the cost of a minimum-sized distributing pipe upon the real-estate frontage, the same way as pavement costs are frequently and sewer costs quite generally assessed. Looking at the subject from quite another point of view, the aim of any city would be to see that an ample supply of pure water, sufficient also in quantity for fire protection and industrial purposes, was available to any and every citizen within the municipal limits, so far as this was not rendered absolutely impracticable by a very scattered population. Obviously, water-works under private ownership cannot accept such a doctrine unless the water rates are high enough to make a profit on well patronized lines of mains sufficient to carry the fixed charges and operating expenses on lines serving only one or two consumers. Such a policy has elements of injustice and is not likely to be permitted where the water consumers are aware that it is being followed. Quite regardless of questions of private and public ownership, if a public water-supply is to be considered as a thing to which every citizen is entitled, quite regardless of the cost of carrying it to an isolated point, obviously the excessive cost thus entailed should be borne by the community as a whole through a general tax rather than put upon that portion of the community which is buying water. When the millennium comes the public water service may be entirely free to individual consumers, the same as is the use of the streets of the city. There is, however, no strict parallel between the reasons for making the two classes of service free, but we cannot go into the details of that point here.

Thus far we have not made mention of any single paper presented at any of the conventions under discussion. We trust that none of the authors of the many papers presented at the three conventions will feel grieved if we single out one paper for particular mention; namely, the one on steel pipe lines, presented last week at Rochester before the New England Water-Works Association, by Mr. Emil Kuichling, M. Am. Soc. C. E., of New York City. The diameter and length of the conduits being called for to provide additional water-supplies in these days of rapidly growing city population and the necessity of going further afield for water make the question of the material which should be used for the water-supply conduits one of great importance. Through its many estimable qualities, particularly that of the relative lightness and consequent cheapness, steel has gained the place as compared with cast-iron during the past twenty years. But, unfortunately, steel is liable to far more rapid corrosion than cast iron, and in this connection the very lightness and thinness of the material renders it subject to quick destruction if rust once gets firm hold of it. Mr. Kuichling presented in an able and interesting manner some of the leading questions affecting the life of steel, as compared with wrought-iron and cast-iron. He also touched upon the important question of deformation.

Finally, we may note one significant lack of the three programs which we have discussed, which lack is also shown by most of the programs of the several water-works conventions year after year. This is the comparatively little time and attention given to everyday questions of water-works operation. These daily questions really are of far more concern to most water-works superintendents than the problems of design and construction, finance, etc., which monopolize the greater part of the time of the conventions. Doubtless, the practical questions of water-works operation would be more frequently and fully presented to the conventions if the men who by their everyday experience know all about them could be induced to present papers embodying their experience. Somehow, this seems to be for the most part impossible. We have mentioned this before. We hope to see a change in the future.

LETTERS TO THE EDITOR.

More About Laying Ordinary Cast-Iron Pipe to Curves.

Sir: Referring to the inquiry of "H. McN." in your issue of Sept. 1, I have found the simplest way of determining deflections and curves to be as follows:

Take two lengths of the pipe being laid, place them in position on level ground, spigot in bell, and throw the end of one until the joint on the inside of a curve is as small as it is possible to get at with a calking tool, about ¼-in. Then, sighting the pipe, measure with a rule the deflection at the outer end of one pipe. This will be the maximum deflection per length; supposing d to represent this deflection in inches and D to represent the maximum degree of curve in degrees; D equals approximately $3\frac{1}{3}$ d. The joint opening, and consequently d and D, vary with different sizes of pipe and usually with the same size from different foundries. I consider it always allowable when the topography of the country demands it, to lay out curves in this manner, provided the depth of lead in the joints be increased on such curves so that on the outside of a joint on a curve it equals that on a tangent. Our maximum curvature on 30-in. pipe has been 12° 30', ranging to 18° on 12-in. pipe. By the use of short lengths, as 6 ft., instead of 12-ft. laying length, D may be made equal to $6\frac{2}{3}$ d.

Yours very truly,
John H. Lance.

Wilkes-Barre, Pa., Sept. 22, 1910.

Some Notes on the "Interval" Method of Stadia Surveying.

Sir: While reading the interesting article on "The Interval Method of Determining Elevations in Stadia Surveys," by Mr. Adolph F. Meyer, published in Engineering News, Sept. 1, 1910, p. 231, the following points occurred to the writer.

Elevations of stadia stations are given to hundredths of a foot but it would seem that large probabilities exist for errors in turning the intervals. On a grassy hillside, newly cultivated land or a field of corn it is very apt to happen that no well defined point will be at the place indicated by the stadia wire. Errors from this source are liable to occur.

The value of 30" as given in the fourth line from the bottom of the right column should be about 0.15 ft.—one-half of 0.3 ft.

In the table at the bottom of the page it is not evident whether corrections for "$f + c$" were made with the vertical angles. That is necessary in comparisons of accuracy. On a stadia station an observation should not be permitted where the bubble is half a division out, as illustrated in the table. Before the vertical angle was read the instrument should have been re-leveled. Then, if, as should always be done, a check azimuth on some prominent object had been taken directly after the back sight, a check on the azimuth could have been obtained and the instrument corrected for azimuth if necessary.

No difficulty has been experienced here with students or others in the use of the "$f + c$" correction when needed and it is preferred to a field determined correction "C" as suggested by Mr. E. M. Douglas (Engineering News, April 28, 1910, p. 483).

As to speed: When the middle wire is set at the proper point on the rod before the eye is removed after reading the intercept, it and the reading of the vertical angle can be done as quickly as the telescope bubble can be set horizontal. Further, the rodman can be sent to a new point as soon as the middle wire is set and thus save for the rodman the time required to set the level horizontal and to turn the intervals.

For the most accurate reductions it may be necessary to use tables or diagrams but for topography sights some form of slide rule is best and very fast. An excellent rule is manufactured by Keuffel and Esser Co. after a design by Wm. Cox. The writer would join in urging a more general use of the stadia method either alone or as a check on tape measurements. In an extensive use in each capacity it has been found of great service.

The loose leaf page shown in Fig. 3 of Mr. Meyer's article, although common, does not seem suitable for a field book. The page is liable to be lost or misplaced and if left in the office the field party is often without needed data. A method which has proved very valuable is to use a bound field note book and to fasten by paper clips a loose leaf and carbon sheet to the back of the two pages upon which the notes are to be recorded. Then carbon copies can be left in the office, so that there is a complete set of notes in the office and also a complete set in the field. The bound book precludes lost or mislaid sheets.

Yours truly,
Charles A. Holden,
Professor of Civil Engineering,
Thayer School of Civil Engineering.

Hanover, N. H., Sept. 3, 1910.

The Design of Arched Dams.

Sir: In your issue of June 9, 1910, p. 678, a letter by Mr. John S. Eastwood was published on curved dams, criticizing the seeming waste of material in the Shoshone arch dam. Mr. Eastwood proposes a plan having probably less than half the material of the one used. There are some features in Mr. Eastwood's design and statements in his letter that will bear discussion.

Mr. Eastwood uses shorter radii in the plan of the arch. This makes the line of thrust at the abutments of the arch on the sides of the gorge nearer parallel with the axis of the stream. At the top of his dam the arched wall is in fact parallel with this axis. This means that the rock would have to be exceedingly solid, or else a chunk would be sheared out of the side by this thrust parallel with its surface and close to the surface. This is a feature of an arched dam that should not be lost sight of. It is more conducive to stability to have the thrust of the arch forcing into the body of the rock sides than to have it approach a direction parallel with the surface. In the latter condition a portion of the side of the gorge might be dislodged.

Mr. Eastwood says:

The arch—the circular arch, the concentric ring arch—when loaded by water pressure is the most perfectly loaded form into which material can be placed for it is the equivalent of a balanced load on a column without length.

Mr. L. A. B. Wade in a paper on curved masonry dams read before the Institute of Civil Engineers, reprinted in part in your issue of May 19, 1910, p. 538, says:

In designing these dams the complex question of the exact stresses that may occur and the assistance that may be afforded by the weight of the wall has been disregarded; as it has been considered that all practical requirements would be met if in theory the dams were treated simply as sections of rigid cylinders subject to exterior water-pressure.

Mr. Wade uses a flat unit stress as high as 20 long tons per sq. ft. or 310 lbs. per sq. in. for the concrete. While he also gives lower units, it is the flat unit that is subject to question. Why should a circular arch be considered as a short column or a rigid cylinder, with no reduction in the unit for unsupported length? The curved shape of the arch does not insure lateral support;

and the water does not supply this lateral support, since its pressure is just balanced by the thrust of the arch. These arches would be in the condition of a thin tube under external pressure, if they formed a complete circle. They are in somewhat similar condition to columns of a length equal to the curved distance from abutment to abutment. It is well known that the collapsing pressure of tubes under external pressure is very much less than that which would give a unit stress to cause failure in a short column. Some formulas based on tests give, for example, in a tube 50 ins. in diameter and 1 in. thick in steel, a collapsing pressure that shows but one-sixth of the compressive strength of the steel.

One of the dams described by Mr. Wade as having been built in Australia has a length at the top more than 200 times its thickness at the top. A column of this length would have practically no strength. Others have lengths greatly exceeding those allowable in columns. It is a good thing that these dams rest on the ground and have the assistance afforded by the weight of the wall, which Mr. Wade disregards.

Basing unit stresses on concrete in the mass upon small cubical specimens, as Mr. Eastwood has done, is not a very safe method of design. Mass concrete is not uniform as small blocks would be. Of course a large factor of safety can be made to cover this, but if the unsupported condition of the wall as a column be ignored, the factor of safety may be eaten up by this condition.

Yours very truly,

Edward Godfrey.

Monongahela Bank Bldg., Pittsburg, Pa., Sept. 7, 1910.

Some Words on the Forest Fires.

Sir: I have read with interest your editorial on western forest fires (Sept. 1), as the Oregon Terminal of the K. L. R. R. is in the woods and surrounded by forest, which has been disastrously burning all around me for the last three weeks (is burning yet to some extent); and the fact that I have taken my men to fight fire, and as near as I can learn we are the only ones that succeeded fully in putting ours out, adds to the interest.

We are here about on the Oregon and California line, about 20 miles from the main line of the Southern Pacific. In the last ten years from two to three thousand acres have been cut over, leaving the "slashings," pine and fir tops, dead stubs, etc. All of this is extremely inflammable. We have no rain practically from May to November; 90% of the small streams go dry; it is one succession of hot days, hot sun, mercury going frequently to 100°, we have very little wind, but a spark in a dry fir will smolder for hours and break into flame with little wind. The first fall rains are liable to do more harm than good, from the fact that they are usually thunderstorms with violent (sic) lightning and the rainfall entirely out of proportion to the rumpus kicked up, with the result that the lightning will set ten fires to the one the little rain puts out.

Forest rangers have charge of the different districts, and the timber owners' associations also have men watching their interests. The country is very sparsely settled and extremely few men are available at the start, and as they have to be shipped in, valuable time necessarily elapses before any adequate force can be obtained.

Near us here, there has been no loss of life and little if any danger of fatalities, although several thousand acres have burned over in the Clover Creek and Pilot Knob fires; both are burning to some extent yet.

I do not pretend to be an expert in the fire matters, but it seems to me the general policy of putting the fires out indiscriminately is all wrong. Suppose it can be kept up for a number of years and have no fires: every year the ground becomes more littered with inflammable pine cones, dead trees that fall, etc. Sooner or later fire is bound to start, and the longer it goes the harder it will be to control it; eventually one will start that can't be controlled and will naturaly burn itself out. Plastering the trees with notices of the stringent state laws against setting fires will not change the status either.

Moderate fires running through this coast timber do not hurt it appreciably. If the foregoing are facts, it would seem that judiciously setting fires and burning over the lands at proper times (which can be done) would be much more efficacious than endeavoring ad infinitum to keep fires absolutely out.

I contend this isn't a case of removing the cause, but removing the effect. Railroads have got to run, and it is either railroads or donkey engines that logging has got to be done with on this coast. The locomotives burn slabs (as far as I know invariably). If you screen the stacks so close as not to throw cinders, you can't make steam. The lumber company near me run through valuable timber owned by the interests I represent; at the psychological moment last fall the woods foreman backfired so effectually as to never have started a fire at all on our land. They have repeatedly set their own slashings on fire, but by patrolling after their trains have never let it get away.

In this sparsely settled country, where even on main roads ranch houses are from 5 to 25 miles apart, a very large percentage of the travel is campers.' I don't care how careful they are or how sure they may be they have left the fire all out, let a wind spring up and blow the ashes left promiscuously, knocking the ashes out of a pipe, a lighted cigar, etc., it would surprise people who have not been out here how little it takes to start a fire, which only emphasizes, to my mind, the impossibility of preventing them. And when put out, the snake is only scotched and not killed.

E. T. Abbott.

Klamath Lake R. R. Co., Pokegama, Ore., Sept. 7, 1910.

Slow Subsidence of Ground in San Francisco and in Baltimore.

Sir: Referring to your comment in the issue of Sept. 8, that it would be interesting to note whether slow subsidence similar to that reported for Cambridge, Mass., by Mr. Frank H. Carter, had been noted elsewhere, I suggest that you publish the following from my recent paper on the Sewer System of San Francisco, Transactions Am. Soc. C. E., Vol. LXV., p. 374:

As City Engineer, during 1900 to 1904, the writer made studies of street subsidence. Careful levels were taken at intervals of one year. The results of these studies for the two years, 1901 to 1903, are shown in brief form in the following table:

SUBSIDENCE OF STREETS IN SAN FRANCISCO.

Location.	Subsidence, in ft. 1901-2.	1902-3.
Davis St., Market to East Sts.....	0.05	....
Jackson St., Montgomery to East Sts..	0.03	....
Spear St., Market to Bryan Sts.....	0.05	....
Mission St., First to East Sts.....	0.05	0.02
Harrison St., Fourth to Seventh Sts....	0.15	0.19
Sixth St., Howard to Brannan Sts.....	0.05	0.05
Sixth St., Brannan to Channel Sts.....	....	0.08

From April, 1902, to April, 1903, there was no appreciable subsidence on Davis St., except a small amount at its intersection with Vallejo St., where the subsidence during the preceding 12 months had been 0.08 ft.

From April, 1902, to April, 1903, there was no appreciable subsidence on Jackson St., except a small amount at Drumm and East Sts., where the subsidence during the preceding year had been 0.06 ft.

From April, 1902, to April, 1903, there was subsidence on Spear St. only at Mission St., where the subsidence during the preceding 12 months had been 0.06 ft.

The subsidence is not confined to the streets alone, but includes adjacent buildings; even those within the area under consideration that are supported on piles 90 ft. and more in length show measurable subsidence.

The lower end of Market St. in 1903 was from 2 to 3 ft. below official grade from curb to curb, the street-car tracks supported on piles having settled with the remainder of the street.

As subsidence of the character here referred to includes every structure in or adjacent to the street, no provision for the support of sewers would be adequate to prevent settling. If the sewers are laid to grade there will be sections dipping below grade after a few years, and the dip will increase until after a time the departure from the true gradient may be so great that reconstruction of some of the down-town sections will be necessary.

Referring to this in the discussion of the paper Mr. Kenneth Allen says in part:

About 1894, precise levels were run in connection with a general topographical survey of Baltimore then being made by Colonel H. T. Douglas, and numerous benchmarks were established. Twelve years later the speaker had occasion to test a good many of these, and, although precise levels were not run, the discrepancies noted were checked so carefully that he was convinced that numerous changes had occurred during the intervening years. This opinion was confirmed by results reached on another division of the same work, where it was observed that the greatest discrepancies occurred where the slopes were most abrupt. Discrepancies of this kind might appear to be due to a difference in the graduation of the rods, but as these had been compared carefully with a standard, the conclusion reached was that this was not the cause. The apparent errors of elevation noted varied up to several hundredths of a foot.

In closing the discussion the writer, in reply to a suggestion that changes in the elevation of the water table and the action of frost might in some cases be the cause of the subsidence, said:

The sinking or settling of certain areas in San Francisco, alluded to in the paper, is a local phenomenon, due entirely to compression of the material on which parts of the city have been built. The writer, while City Engineer, commenced the work of precise leveling which is being extended to all parts of the city. In connection with this work, tests were made to find whether certain low streets, the surface of which had once been at official grade, were still settling, or whether they had come to rest. It was found that some of them were still sinking. Neither frost nor change of ground-water level has anything to do with this phenomenon.

C. E. Grunsky.

San Francisco, Cal., Sept. 14, 1910.

[When settlements are so distinctly localized as at Cambridge, in a limited area too large to be affected by any single condition of surface loading, the causes are not obvious. Apparently there is somewhat similar localization in San Francisco, though Mr. Grunsky's figures do not make it clear whether specific areas are affected as at Cambridge.—Ed.]

Increasing the Speed of Driving Railway Tunnels.

Sir: The great majority of railway tunnels are comparatively short and are often driven without any machinery at all. On new lines where the tunnel sites are located far from the supply base, it is common practice to sublet the tunnels under 600 ft. in length to gangs of hand drillers. The hand drillers aim to break the rock with as few holes as possible and to that end spend a great deal of time and patience in chambering out the drill-holes so they will take a heavy blasting charge. This method of driving leaves a rather irregular tunnel section, but is economical, and the costs per foot for standard single-track section will often run as low as $35.

If the rock is very hard, however, it is difficult to get hand drillers to take the work at a reasonable price. In such cases steam drills can be used in short tunnels at a saving over air drills. A steam drill uses only half the fuel that an air drill requires and causes no inconvenience until the drive is 300 or 400 ft. from the portal. In using steam the benches are carried close up with the heading, the heading and top bench being drilled with one set of the columns. The exhaust steam is piped outside, and the heat and what steam escapes roll out along the top of the tunnel, creating a good circulation of air. It is always cool below the level of the heading.

In driving long tunnels the question of speed becomes of paramount importance because a large capital outlay is tied up while the work is in progress. In most of the big tunnel undertakings, costs per foot have been sacrificed to speed. The following method of tunneling which I am suggesting is intended to duplicate the great speed records which have been made in the long Alpine tunnels but at a greatly reduced cost per foot. There are in this country several long tunnel projects that would cut off thousands of feet of train climb, which are held in abeyance because the traffic would not justify the high cost that modern high-speed tunneling exacts.

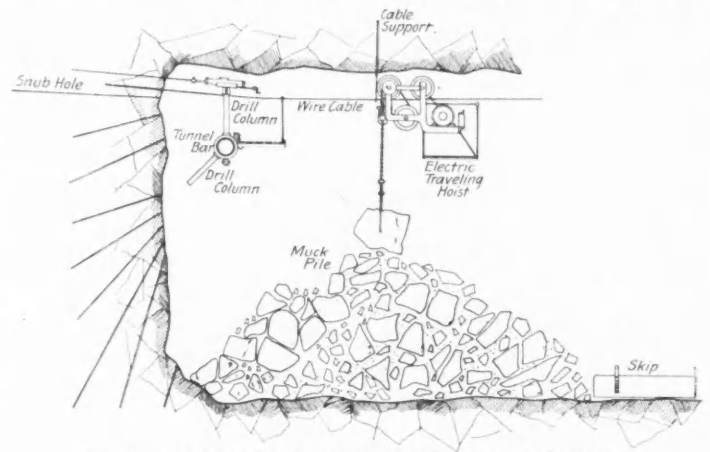


FIG. 1. PROPOSED METHOD OF DRIVING HARD-ROCK TUNNELS WITH FULL-SECTION FACE.

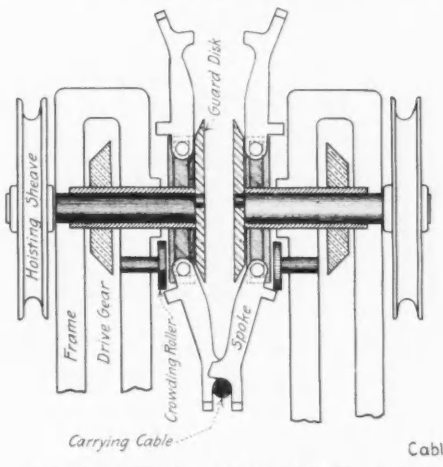
The idea is to do away with the heading entirely and advance the full section of the tunnel as a square face, the cuts into the solid being made with long slant holes which break their burden clean with moderate charges of explosive and do not jolt and shatter the surrounding rock. The main disadvantage of the heading system of driving tunnels is that, owing to the confined space, the cut holes have to be put in nearly straight and blasted with a heavy charge of high-grade explosive. The jar of the explosion has a tendency to open up the cleavages and rifts in the rock adjacent, rendering the roof drummy and treacherous.

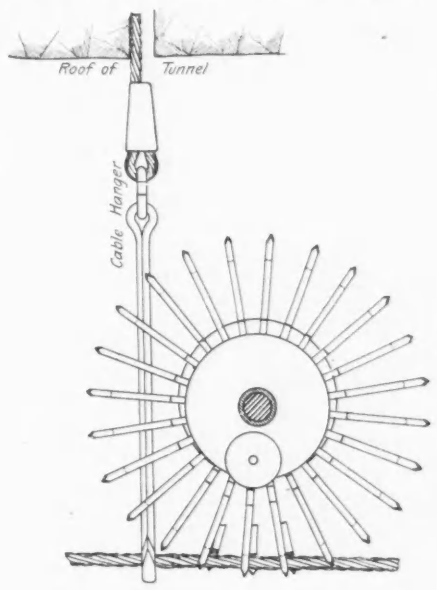
The sketch Fig. 1 herewith illustrates the proposed method of drilling the face. The battery of drills is mounted on short columns swung from a heavy tunnel bar. The tunnel bar would be handled by an overhead hoist traveling on a cable suspended from the roof. The muck could be loaded in skips either with a small shovel or by hand and transferred and dumped into the cars by the overhead hoist. The overhead cable would be suspended on eyebars spaced about 30 ft. apart, the eyebars to hook into wire-rope hangers securely an-

chored into drill-holes drilled in the roof. The rope is to be wedged against the side of the hole by driving a chain of wedge-shaped links alongside, each link being driven solid with a long tamping bar.

The traveling hoist would straddle the cable, the carrying sheaves being made as also shown in Fig. 2. Two oppositely mounted disks carry a number of spokes pivoted to their periphery, which a pair of crowding rollers presses outward in the lower arc of their travel so their treads interlock over the cable. When the sheaves approach a hanger, the bevel edges of both spoke and hanger cause the spokes to crowd in fore and aft of the hanger.

A tunnel bar as shown will easily mount ten or twelve drills, and could be swung in position and jacked up in a very few minutes. A possible improvement in mounting the drills would be to use sleeves on the column arms actuated by an air piston, the drills clamping on the sleeve in place of on the arm direct. Mounted in this manner the machines could be shifted laterally about 4 ins. without affecting their alinement, and in changing steel the machines could be shifted so the steel can be easily withdrawn from the hole. One of the greatest difficulties to overcome in turning out a big footage with a drill is the time lost in crooked or fissured holes, and the best way to minimize this is to start the holes with cruciform steel of slightly less diameter than the cutting bit and run the hole three or four feet with these large steels. This method requires the machine to be swung out of the way to change a steel hence the possible advantage of a sliding sleeve. Having the entire battery of drills concentrated in one place one blow pipe would blow the holes after a run with hardly a second's loss of time; a couple of air hammers could also be used to cut hitches so the slant holes could be collared without loss of time.





Cableway and Carrier.

FIG. 2. DETAILS OF PROPOSED CABLE AND CARRIER SYSTEM FOR MUCKING.

Usually it will not pay to break more than seven or eight feet of ground per cut; more than this would bank up the muck too high at the face. The blasting of the holes will require considerable time until the work is thoroughly systematized and the men become accustomed to the routine. In general it pays to drill holes of large diameter and load with a moderate grade of explosive, which is not only safer to handle but will throw the rock out better than the high-grade explosive.

The best system of blasting is with delayed-action electric exploders. These permit of firing the entire round at one operation; at the same time the squaring holes follow the cut and relieving holes in their proper sequence. The ideal electric exploder would be one in which the delayed action could be adjusted to intervals as small as a hundredth of a second. With a small time interval one shot couldn't pull back on another hole and carry unexploded dynamite into the muck pile and the shots would follow each other in such quick succession that the muck would be heaved out en masse. It would seem as though this delayed action could be obtained by jacketing the platinum bridge in the exploder with a non-conductor of heat and leaving a stem of metal project through into the fulminate.

To advance the tunnel at the rate the Alpine tunnel headings have been driven, i. e., about 600 ft. per month, it would be necessary to drill and blast a cut every eight hours. As perhaps four hours per shift would be consumed in rigging up, tearing down, and blasting the round, the 300 odd feet of holes required must be drilled in less than four hours. This would require from eight to twelve machines. [Four drills only are used in each heading of the Loetschberg Tunnel, where rates of progress of 600 to 800 ft. are maintained month after month.—Ed.] Each round of holes would throw out from 120 to 140 yds. of muck.

The arrangement of a hoist traveling on an overhead cable lends itself admirably to the handling of the muck. Where the rock has a tendency to break up fine it would be possible to use a small electrically operated shovel for loading the skips. Where the rock can be broken in fairly large masses the skips can be loaded by hand almost as fast as they can be handled by the hoist. The arrangement of holes shown would tend to throw out the rock mostly in good-sized chunks that can be rapidly loaded. The large masses can be pulled out of the pile with tongs or chain and transferred direct to the rock train; smaller pieces can be barred and rolled down into skips placed at the foot of the muck pile.

Geo. C. McFarlane.

Iron Bldg., Denver, Colo., Aug. 9, 1910.

Brick Pavements Given Scant Consideration by the International Road Congress.

Sir: I read with considerable interest the account of the Second International Road Congress by Mr. E. L. Corthell in Engineering News of Sept. 1. I do not know all of the personnel of the American delegates who attended that convention, but so far as I am aware no delegate was sent from any of our states or from the United States Government. If such had been the case the country would have been fairly represented and the condition, progress and advancement of road building in this country would have been given fairly to the convention. On the face of the reports from the convention such was not the case, and a fair conclusion leads one to believe that the American delegates allowed their personal interest to actuate them in all that was said and done, or they were unable to impress themselves upon the convention.

The improved streets and roads in this country that have wearing surfaces of vitrified brick exceeds in square yards by two fold more than the combined wearing surfaces made of any other material. In the five progressive states in the middle west wherein there is a larger proportion of mileage of improved streets and roads than that of any other portion of this country, the brick wearing surface exceeds that of any one other material more than ten fold. [We know of no statistics bearing out these assertions, but do not mean to say that Mr. Blair could not produce them.—Ed.]

If this world's Congress is to be of any value at all to civilization, it should be made to disclose exhaustively a study of the condition it pretends to meet.

Under Article V. of Final Conclusions concerning the choice of surfacing materials may be found, though not stated, extraordinary deductions in favor of vitrified brick as a wearing material as compared with the merits and demerits of other materials.

Take Section 1 reading as follows: "Macadam causes dust and mud, is expensive to maintain, and is suitable in large cities only for streets where the traffic is not very great or heavy." The brick wearing surface causes neither dust nor mud as it does not originate them. It is not expensive to maintain; it is suitable in large cities for streets where the traffic is heavy. For ten years the street through Waterbury, Conn., leading from the railroad to the Chase Rolling Mills, has borne a greater tonnage with less tractive resistance than any street in the City of New York, and the vitrified brick street has not been subject to repair for ten years. Of course the macadam street is properly said to cause dust and mud, and hence is very unsanitary. Properly constructed vitrified brick streets are of the most sanitary character.

Section 2 reads as follows:

The experimental work carried out in recent years with macadam improved by using a bituminous or tarry coating of binder must be continued in order to determine the best methods of utilizing this kind of construction under varying conditions, so that this question may be submitted again at the next conference.

The foregoing conclusions with reference to bituminous binders very properly puts them in an experimental class. A properly constructed vitrified brick street no longer belongs in the experimental class. Numerous streets and roads in this country subject to both light and heavy traffic have not been subject to a single penny of expenditure for repairs and such pavements do not indicate that they will be the subject of repair for years to come. They cannot, therefore, be ranked with an experimental class, rather they afford ideals for the world.

Section 3 reads as follows:

Stone pavement has great qualities of resistance and durability. Its maintenance is easy and economical; it produces hardly any dust, and is suitable where there are tramway tracks.

It is entirely within the bounds of truth to say that while the stone pavements were given recognition they were not given the recognition due them. In fact, because of their inherent merit as a road wearing material, in so many cases, like vitrified brick, they are not given the advantage in construction they deserve. Few of our granite and stone pavements are constructed with any intelligent ideas as to their purpose in use, or with a view of permanent maintenance without cost.

Section 4 reads:

It [stone] should be adopted in thoroughfares whenever noise is of little consequence or when wood or asphalt surfaces are not suitable. It should consist of setts [blocks] regular in shape, durable, but not slippery, wearing evenly, and laid upon a foundation and with close joints.

These blocks of stone should be laid upon a foundation cushion of 2 ins. of sand, the interstices should be filled with cement and then they will wear evenly in spite of whatever uses they may be subjected to; and it may be added that when so laid the noise will be of little consequence to anyone because there will not be much of it if municipalities will follow the suggestions aforesaid.

In complying with the request of the Congress in Section 5, which reads, "The Congress desires to see the trials of small sett pavements continued wherever local circumstances and traffic conditions permit," they will find that their stone pavements will come much nearer reaching all of the splendid qualities to be found in the properly constructed vitrified brick pavements and can be adapted in many places with very satisfactory results.

Section 6 reads:

Wood paving is noiseless; not slippery if kept clean; it stands very heavy traffic. The use of it should be extended even to thoroughfares through which run tramway lines.

Wood paving is noiseless, so said. I do not know the kind of wood in the use of which that opinion is evolved. It is putting it mild enough to say that that kind of a statement should be followed by information as to whereabouts. It certainly is noisy beyond that of asphalt or bituminous pavements when the pavement is dry. It even exceeds the noise of a properly constructed brick pavement when the wood pavement is dry. As to the brick pavement the average noise is no more than that of the wood pavement. The brick pavements are not slippery, wet or dry, clean or dirty.

Section 7 reading as follows:

The respective advantages of soft and hard wood blocks must be a subject of discussion at a forthcoming Congress.

This has not much relation to the other kind of pavements, and therefore may be passed by with a remark that the advantage as between the two, hard and soft, is entirely a commercial question.

Section 8 is:

Asphalt pavements should be recommended owing to their good qualities from the hygienic point of view and their ease of cleansing and of repair. Their surfacing is almost noiseless and produces but little dust, but it is unsatisfactory adjacent to tramway rails.

Regardless of cost, a properly constructed vitrified brick street pavement meets every desirable point that is found in the asphalt pavement.

Section 9, on asphalt, reads as follows:

There is opportunity for its use in fashionable thoroughfares where the traffic is not severe, where there are no tramways, and where the grades are very moderate.

The suggestion that there is opportunity for its use in fashionable thoroughfares indicates that to a certain extent it meets esthetic ideas, but in that respect alone does the asphalt pavement excel that of brick. No more beautiful, no more handsome, no more fashionable residential road can be found any place in the world than that of Wade Park, Cleveland, Ohio, and it is not extravagant to say that its streets surfaced with vitrified brick are an adornment to the locality and appeal in the very highest degree to the conception of artistic beauty possible with any like locality.

It is but fair that this much at least be said in behalf of this country, as well as to sustain its reputation of both advance thought and practice in comparison with the rest of the world.

Will P. Blair,
Secretary National Paving Brick Manufacturers' Association.

Board of Trade Building, Indianapolis, Ind., Sept. 14, 1910.

[It is not so surprising as Mr. Blair thinks that more prominence was not given to brick pavements at the International Road Congress. Most of the delegates went to Brussels to discuss macadamized roads of various types rather than city pavements of any kind. Of the delegates at all concerned with city pavements relatively few, if we are not greatly mistaken, possessed either experience with or interest in brick as a paving material. Nevertheless, we gladly publish the letter, as showing up the points for brick that occur to a wide awake experienced promoter of a paving material which has most assuredly made a place for itself in many cities and on a few country roads.—Ed.]

The Chicago Stock Yards Water Purification Litigation.

Sir: The filtration plant at the Union Stock Yards in Chicago, built in 1907-8 for the purpose of purifying for stock-watering purposes the waters of "Bubbly Creek," has been the cause of litigation in the quite recent past. In the Engineering News of Sept. 8, 1910, there appeared an article bearing upon the case of the City of Chicago vs. The Union Stock Yard & Transit Co. As the writer was in a considerable measure responsible for the acceptance of this filter plant by the Stock Yards Co., it is his purpose to discuss as briefly as possible the chief issues which developed during the trial of this case.

Events Leading Up to the Trial.

In 1907 the Union Stock Yard & Transit Co., riparian owner of the South Fork of the South Branch of the Chicago River, this fork being sometimes referred to as Bubbly Creek, entered into a contract with the Norwood Engineering Co. for the construction of filtration works. These works were to be located on the company's land and on the south bank of Bubbly Creek.

The contract between the Stock Yards Co. and the Norwood Co. stipulated, among other things, that the filtration works should be capable of delivering at all times an effluent of a specified degree of purity. The contract was submitted to and approved by the local health department. The Stock Yards Co. furthermore applied for and obtained from the city of Chicago a permit for the erection of this plant and for the construction of the intake in Bubbly Creek.

The filtration works were erected, and after a careful test had been made of their efficiency it was decided that the contract guarantees had been fulfilled and the contractor was paid in full. In September, 1908, the works were placed in official service, and from that time on the filtered product was delivered into the reservoir of the Stock Yards Co. and distributed therefrom to the various pens for consumption by the live stock.

In February, 1909, some four months after the filter plant began to supply water to the yards, the City Council of Chicago passed the following ordinance:

No person or corporation shall use water taken from the Chicago River or any of its branches, or from any body of water within the city of Chicago, which is stagnant, or in which refuse, waste, garbage, sewage or any other material tending to destroy the purity of the water has been dumped, placed or discharged, or in any manner found, for domestic purposes or for watering livestock, or for use in the preparation of meats, poultry or provisions for the market which are intended for consumption, or for any other use whatever which endangers the public health. But nothing in this ordinance contained shall be construed as limiting or prohibiting the rights of persons or corporations to use such water as a means of supplying motive power for mechanical purposes.

Following the passage of this ordinance the city brought suit against the Stock Yards Co. on the ground that the company was violating the above ordinance. The trial of the case was begun on Jan. 31, 1910, and altogether some 1,000 pages of testimony were given before the Hon. John C. Scovel, Judge of the Municipal Court of Chicago.

Line of Defense Advanced By the Stock Yards Company.

It is well understood and recognized by everybody in the sanitary world that the great majority of all public water-supplies of this and other countries are derived from sources which are, at one time or another, subject to a varying degree of pollution from the sewagic wastes of the development of mankind. In its opening, therefore, the company claimed that the ordinance referred to was oppressive, unjust and unreasonable, and not a proper exercise of the police powers of the city; and that if enforced according to its terms it would prohibit the city of Chicago from taking its water-supply from any known source in the vicinity of Chicago, including Lake Michigan, its present source of supply. It was claimed that the ordinance deprived the company of its property without due process of law, and that such action was in contravention of the constitution of the State of Illinois, and the constitution of the United States and the 14th Amendment thereof.

The Stock Yards Co. set out to show that, even though the waters of Bubbly Creek were initially foully polluted, by the process of purification employed at the stock yards filter plant the water as it left these works was a safe and proper water to use for stock-watering purposes. A stipulation of facts was drawn up and with some exceptions agreed to both by the corporation counsel's office and the attorneys for the Stock Yards Co., and in Section 6 of this stipulation the statement occurs:

That the effluent coming from said filtration plant was, on Oct. 1, 1908, and thence hitherto, and is now, used by the defendant for watering live stock in its stock yards, and that there is no connection with said plant for distributing said water for domestic or drinking purposes.

From a sanitary standpoint the whole case rested upon whether or not the waters of Bubbly Creek, primarily foully impure but purged by the purification process of all their visible impurities, and rendered clear and sparkling, but still containing in a dissolved state about 50% of the organic matter which the raw water contained, in the practical absence of bacterial life, and in the absence so far as bacteriological tests now can show of all bacteria commonly known as disease-producing forms, could still be considered a satisfactory water with which to water live stock.

This filter plant was not constructed for the purpose of producing from Bubbly Creek a water-supply for consumption by human beings. For some reason or other, as the trial progressed the city pushed this phase of the question more and more to the front, until ultimately it was made to appear that, among other things, the company had installed this plant to render the waters of Bubbly Creek safe for human consumption, and that one of the final objects the Stock Yards Co. had in view was to substitute for the city water the effluent of the filter plant to be used throughout the yards for human consumption, as well as for watering livestock.

Why the Stock Yards Filter Plant Was Built.

The prime reasons why the Stock Yards Co. decided to construct the filtration works under discussion were that the contractors, the Norwood Engineering Co., and their associates, after quite an extended series of tests with Bubbly Creek water, satisfied themselves that they could render those waters safe for stock-watering purposes, and so notified the management of the Stock Yard Co. Furthermore, they offered to enter into a contract with this company for building such works, the acceptance of which was to depend, among other things, upon the ability of the works to deliver a product which "shall be applicable to all uses for which the city water is applicable." They furthermore guaranteed that the cost of production of such water should not exceed 20 per million gallons, interest on the investment and depreciation not included. As this figure was several times smaller than the cost of city water, in view of the guarantees given by the Norwood Engineering Co. the Stock Yards Co. decided to sign the contract. The writer has positive knowledge, however, that the company never intended to use the filtered product for other than stock watering, sprinkling, flushing and steam raising purposes.

There was another feature which probably had something to do with the Stock Yard Co.'s decision. There is in the yards a reservoir having a capacity of some 3,000,000 gals. The demand for water, particularly on certain days, is so great that without this reservoir, and with the existing connections with the city mains, it is impossible to supply the demand. It was a much discussed question as to whether, under extreme conditions, the city supply would not fail, and such failure in case of a fire occurring coincident with the heaviest draft would likely be followed by disastrous results. With the filter plant running to its full capacity of 5,000,000 gals. per day such dangers would be obviated.

Shall a Private Corporation Control a Public Water-Supply?

The view was held by the city that to entrust to a private corporation not primarily engaged in the water business so important a function as the control of a public water-supply is unsafe and improper. It is necessary in this connection to draw a fine line of distinction with respect to what constitutes a public water-supply. It has already been pointed out that the object of the Stock Yards Co. was not to use the filtered product for human consumption, but for stock-watering and mechanical purposes.

Speaking broadly, however, and eliminating this point, it would seem that a corporation so large and so responsible as the Union Stock Yard & Transit Co., which chose for economical among other reasons to enter into the water business to the extent of constructing a filtration plant having a capacity sufficient to supply water to a community of 50,000 people, might properly be considered to be practically engaged in the water business.

It is furthermore to be pointed out that the Stock Yards plant has always been operated under the expert supervision of a man well versed in the art of water purification, and that he had competent assistants to take his place in case of absence or illness. The testimony given at the time of the trial is sufficient to show that the operation of the plant was as nearly automatic as it is possible to make such a plant, that all the pumps, chemical dosing apparatus and other mechanical appliances were provided in duplicate to guard against complications in the event of a breakdown. The successful operation of this plant day after day for months on end, supplied evidence of a sufficiently conclusive character to convince such eminent specialists as testified on behalf of the Stock Yards Co. that the element of danger from mismanagement had been minimized to the greatest practicable degree.

It is further to be pointed out that the plant was always under the eye of the local health department. Not only were samples periodically collected for analysis by representatives from this department, but an analyst was kept at the plant for the purpose of watching the procedures of operation and at frequent intervals to make analyses of the filtered water, the results of which were reported direct to the health department. It would not be possible for any water purification plant to be kept under a tighter rein of control.

The Reliability of the Process and Management of the Filter Plant.

The process of purification employed at the Stock Yards filter plant, in all its essential details, is the same as that successfully employed in scores of cities in this country for the purification of polluted surface waters. It consists in the addition to the raw water of sulphate of alumina followed by a period of coagulation and sedimentation of some three hours. Before the water goes to the filters it is treated with the powerful germicide, hypochlorite of lime, about which so much has been favorably written in the last year or so. Finally the partially clarified and sterilized water is filtered through sand filters of the mechanical type, and the finished product discharged therefrom into the clear water well, flowing thence to the reservoir in the yards.

Since March, 1908, the writer has been in close touch with the performance of this plant, and has spent many weeks on the ground studying every detail of its operation. Under the circumstances he can do no better in discussing this point than to quote from his testimony given at the time of the trial, as follows:

I am in a position to testify that all parts of the plant are capable of being kept under complete control, and that the process of purification is a thoroughly reliable one.

There is no opportunity for an interruption in the pumpage of raw water into the plant. The centrifugal pumps which raise the raw water into the mixing channel are provided in duplicate, one being used all of the time. When that pump requires repairs the other pump is thrown into service.

The chemical apparatus is also arranged in duplicate so that if any part of it gets out of order it can be easily repaired, at the same time using its mate. The devices for measuring the flow of chemicals, the facilities for notifying the operators if any interruption occurs in the flow of chemical solutions are sufficient, to my mind, to insure a steady, constant, accurate and reliable flow of chemical solutions into the water at all times.

The filters themselves have never been run to their utmost capacity except during the test. They are capable of yielding more satisfactorily filtered water than they are called upon to yield now. They are thoroughly reliable, and even more so because of the fact that the practical sterilization of the water before it reaches the sand layer, leaves for the filters only the work of mere mechanical straining.

I have studied with much care the modus operandi of Mr. Jennings, and am free to admit that I know of no plant of its size in the United States that is under such expert supervision as is this plant. It stands alone among small filter plants.

Studying Mr. Jennings' methods and those of his assistants, I have no hesitancy in saying that since I first became connected with the operation of this plant with him I have discovered no flaw in his methods. I consider the process and the plant thoroughly reliable, and capable of turning out a first-class and safe water at all times.

Another witness, Mr. George W. Fuller, testified on this point as follows:

I believe that this plant here at the stock yards is rather an unusual one in that respect, and for its size is more properly operated from a technical standpoint, with a view to safeguarding the quality of the water, than any other plant of its size that I know of. It has two technically trained men in charge, and there is always one present from the city Board of Health, making three men of this class for a 5,000,000-gal. plant. That is unusual. With its chemicals for the purpose of coagulation, and for the purpose of oxidation and sterilization, it is perfectly feasible, with this degree of technical supervision, to add them in a way to bring about thoroughly reliable results, not only as regards the general proposition, but in particular as to this plant at the stock yards.

The possibility of trouble coming from carelessness in management is well recognized in the case of all water purification plants. In the case of the Stock Yards plant this danger was reduced to a minimum. The hypochlorite process of sterilization, which forms an integral part of the purification system, affords an exceedingly economical and harmless means of removing virtually the last fraction of potential danger from this water. The likelihood of interruption in the application of this

germicide, upon which it is admitted the safety of the water very largely depends, was shown to be practically nil.

Is Water So Badly Polluted as That of Bubbly Creek a Proper Source of Drinking Water?

This is a much mooted question among sanitarians, but one which is far from being settled. Where the line may be drawn between polluted waters which may be rendered safe by filtration and sterilization processes and those which may not is beyond the writer's ability to indicate at this time.

It is true that the waters of Bubbly Creek are much fouler than those ordinarily encountered in problems of water purification. The difference between the purification of such water and waters of far greater purity is, from a broad, practical standpoint, only a matter of degree. The raw waters of the Allegheny River at Pittsburg sometimes contain as many bacteria per cubic centimeter as are found during the worst periods in Bubbly Creek. The waters of the Seine at Suresnes and Ivry, and of the Norder Elbe below Hamburg, where Altona obtains its supply, are in more or less the same class of foully-polluted waters. At all of the above-mentioned places the waters are purified and made perfectly safe for human consumption, even though they still contain in large quantities organic matter of sewage origin.

Then again it is necessary to return to the statement that the Stock Yard filter was not constructed to furnish from Bubbly Creek a water which would be used for human consumption. On the question stated in the above heading, however, the city took the negative side. To the writer it would appear necessary only to recite under the next two headings certain testimony presented during the trial, all of which bear directly upon the effect on health of the constituents still remaining in a water, primarily foul, but rendered physically satisfactory and freed from disease-producing germs.

The Effect on Health of Organic Matter and Its Decomposition Products in Drinking Water.

The city endeavored to bring out the fact that, even though by the purification process the raw water was made pleasing to the eye and rendered practically free from bacterial life, the filtered product still contained dissolved organic matter in such quantities as would be likely to exert a toxic influence on persons drinking such water continuously.

Anticipating that this contention would be raised, the experts for the Stock Yards Co. considered carefully all of the available evidence bearing on this question. As a result the conclusion was reached, which conclusion was substantiated by the writings of numerous authorities on the subject, that in the absence of bacterial life organic matter is not injurious to the public health when consumed in such quantities as were found in the filtered water at the Union Stock Yards.

Without going into unnecessary detail with respect to this claim, it would be well to point out that so far as the writer is aware, there is no positive evidence available at this time to support such an assumption. One important witness, Prof. John H. Long, a noted toxicologist, who was a member of the so-called referee board, a commission appointed by President Roosevelt to study the effect of food preservatives, testified in connection with this question that:

From my experiments I concluded that substances which are to be considered harmful are not present in the filtered water.

In his cross examination this point was again brought up as follows:

Mr. Childs.—What limit would you consider as the amount of organic matter that might be in water which was practically sterile so far as bacteria are concerned?

Prof. Long.—I could not give any exact limit, but undoubtedly a larger amount of organic matter than we have in this case would still be within a reasonable limit.

Mr. Childs.—Do you consider that organic matter that comes from sewage is not more harmful than water which has organic matter in it that comes from decayed vegetables?

Prof. Long.—When you are talking about water purification, there is absolutely no difference in it. The protein matters are the same. It is a well-established fact that they yield the same decomposition products.

Much more evidence of this character could be here cited. It is deemed sufficient, however, to give the following references which bear upon this point and in which the same views, practically speaking, are expressed, namely, Report of the National Board of Health, Senate Doc. Second Session, 47th Congress, 1882-1883, Vol. I., part 2, p. 201. Second Report of the Royal Commission on Sewage Disposal of Great Britain, 1902, page 27, Report of the Massachusetts State Board of Health, 1890, page 537. Still further evidence is furnished in the testimony of Count Max Le Couppey de la Forest, Secretary of the Commission d'Etudes des Eaux, of Paris, France, given in 1903 in the case of the State of Missouri vs. the State of Illinois and the Sanitary District of Chicago, pp. 4829 et seq. Here he makes the statement that the effluent of the sewage farms of Paris is drunk by all of the workmen who live on these different farms. There are perhaps 200 or 400 of these workmen living in this country. They drink only the waters of the effluent, and we have a chemical survey of these men. All of the physicians are connected with the city of Paris and must give weekly reports upon the sanitary conditions of all these men, all these workmen, and since these different irrigation fields have been in operation we have had no typhoid fever, or cholera, or dysentery, or diseases which come from water.

Suitability of the Filtered Product for Stock-Watering Purposes.

The city did not attempt very strongly to force to the front the viewpoint that the water as it left the filtration plant was unsuitable for stock-watering purposes. It did claim, however, that after it left the plant, and was discharged into the open reservoir in the yards, the opportunities for subsequent pollution in this reservoir were such as to make the water unsuitable.

Why the city saw fit to raise this contention is somewhat of a problem. It is true that the surroundings of this reservoir are capable of improvement, and the writer understands that such improvements are contemplated by the Stock Yards Co. It seems somewhat strange that this reservoir should have been drawn so prominently into the case for the simple reason that for many years it has been used as a storage reservoir for city water, and during that time its surroundings have been practically the same—yet no objections to it have been raised by the city health authorities, so far as the writer is aware. Certainly if the surroundings of the reservoir are unsanitary now they must have been equally so before the filter plant was built. In any event the efficiency of the purification system cannot be criticized because of any shortcomings on the part of the storage reservoir.

Even assuming, for the sake of argument, that the filtered water becomes contaminated in the reservoir it is well to consider that in practically all places where stock is kept the conditions for maintaining a pure water-supply are no better and frequently are far worse than those which exist at the stock yards, or did exist at the time of the trial. From the standpoint of the idealist, there is no question that conditions about the reservoir could be materially bettered, but from the practical standpoint it is well to understand that it is impossible long to maintain water in anything like the condition of purity in which it is originally supplied to live stock. It is almost inevitable that it will promptly be self-polluted in the troughs.

Suitability of the Filtered Product for Human Consumption.

It is perhaps unnecessary to reiterate the statement that this filtered water was not intended to be used, nor was it generally used, for human consumption. The company always believed that the aesthetic objections to it would be sufficiently strong to prevent such use, even though it might be perfectly safe for all uses. The testimony given in the case disclosed the fact that but one man drank the filtered effluent continuously, namely, Mr. C. A. Jennings, Superintendent of the filter plant, and in the opinion of Prof. John H. Long, of Northwestern University, whose testimony on another point has already been quoted, such use of the water by Mr. Jennings was "quite sufficient to show that there was no harmful quality left in that water." That the impression should be scattered broadcast that the filtered product was used generally by the Stock Yards for drinking purposes as a substitute for the city water-supply is doing the company an injustice, and requires this explanation.

Attention is also directed to the evidence given by Prof. E. G. Smith, of Beloit College, who in this connection testified on cross-examination in this case as follows:

Mr. Childs.—If you were to select a water-supply for a community and have the two sources of supply, Bubbly Creek, with this filtration plant as operated by Mr. Jennings, and Lake Michigan, with the water that we could get from Lake Michigan, in the shape that it is in, would you recommend the Bubbly Creek supply instead of the Lake Michigan supply?

Prof. Smith.—I should say that it would be more safe and usable than the other. Yes.

Why the Use of the Filtered Water for Stock Watering Purposes was Discontinued.

No decision was rendered in the case under discussion for the reason that the use of the filtered water for stock-watering purposes was discontinued by the Stock Yard Co. in April of this year. This move on the part of the company was occasioned by complaints by the live stock shippers that the animals drinking the filtered water made less gain in weight than when city water was supplied to them, and this question of weight of course affected the price the shippers received for their animals. The Stock Yards Co. claimed that its own tests and those made by disinterested parties did not support this claim, but moved by threats on the part of the shippers to send their animals to other cities, and fearing its business would be effected through prejudice, the demand of the shippers was acceded to. The suitability of the filtered water was therefore never definitely decided. From the evidence given at the trial, and such other information as is possessed by the writer, it would appear that the Stock Yards Co. was in the right, and that the filtered water was at all times suitable for the purpose for which it was intended, namely, the watering of livestock.

Very truly yours,

Geo. A. Johnson.

170 Broadway, New York, Sept. 23, 1910.

Telephone Dispatching and Train-Order Signals for Electric Railways.

The methods of handling and regulating traffic on electric interurban railways have been developed upon the lines of the train-dispatching system which for years has been (and still is) characteristic of railway operation in this country. There are two special points of difference from steam railway practice, however. In the first place, the telephone is used instead of the telegraph.* In the second place, comparatively few of the intermediate stations have operators, so that there is less control of the train crews. Under this dispatching system, of course, the trains are run by schedules and train orders, reporting to or receiving orders from the dispatchers at stations, meeting points, etc. The rules and regulations affecting train operation are in general similar to those of the American Railway Association. The American Street & Interurban Railway Association has been working for some time upon the formulation of a set of standard rules, based upon those of the former association.

Telephone train-dispatching as applied to steam-railway service has been described in our columns, and we give herewith some particulars as to the equipment and methods in use for electric railways. For much of this information we are indebted to the Western Electric Co., of Chicago.

A metallic-circuit telephone line extends along the railway and is led to the dispatcher's office or to the switchboard of a private exchange. The trainmen call up the dispatcher at starting points, passing sidings, etc., according to the rules, and can call up for special instructions at different points along the line.

On electric railways where traffic is not specially heavy, an ordinary magneto-ring telephone system is usually employed, standard wood-case instruments being used, placed in waiting rooms, booths or boxes along the line of the road. The system is frequently employed on pole lines carrying 500-volt feeders and sometimes on the same poles with the high-tension transmission line.

There is no known system of protection which will render a telephone line absolutely safe when strung on the same poles with high-tension wires. The only way in which such safety can be obtained is by building a separate pole line for the telephone system, and this is usually out of the question. In many cases, when the power wires are of not especially high voltage (say from 500 volts direct-current to 2,200 volts alternating current), no special protection is employed beyond the ordinary lightning arresters and fuses. But of course the telephone lines are separated as far as is practicable from the power wires.

With higher potentials, ranging from 6,600 to 11,000 volts alternating current, it is generally advisable to install further protection merely to prevent the telephone users from getting static shocks from the apparatus. When these high potentials are paralleled by a telephone line, very frequent transpositions of the telephone line are necessary in order to eliminate induction noises as far as possible. Explosive fuses are also inserted in the drop wires leading from the telephone line to the instruments and lightning arresters are used in connection with these. In case of a direct cross with the high-tension line, these fuses will shatter and let the drop wires fall free, thereby cutting off all telephone sets from the line. The telephone wires for such lines are mounted on high-voltage insulators, rated at about 10,000 volts. These are for the purpose of preventing static flashing-over, which would make the telephone line very noisy.

It is also a frequent practice with these lines to mount a platform, below the telephone set, on high-voltage insulators, so that the man using

*It is to be noted that many steam railways are now adopting telephone communication.

the telephone is fully insulated from the ground. Insulating transformers especially designed for these conditions are also used between the telephone line and the telephone instruments, their object being to insulate the telephone apparatus from the telephone line, thereby rendering the chances of shock much less. It is not possible, however, to use many of these transformers on the line, since it is then difficult to ring with them and they affect the transmission of speech over the line.

In connection with outfits fixed in place as described above, portable telephone sets, carried in the cars, are sometimes used instead of instruments mounted on the poles along the line. Taps are brought down the poles at regular intervals to contact jacks which are mounted permanently on the poles. The portable set has a corresponding plug attached to it and when the car stops at one of these points the conductor can connect to the line and get into direct touch with the dispatcher or central office. The Syracuse, Lake Shore & Northern Ry., which operates approximately 250 miles of line, has adopted a portable telephone set of this kind.

These portable instruments have been particularly designed so as to be light in weight, substantial in construction, compact, and with all parts readily accessible. The instrument used by the above road is $11\frac{1}{2} \times 10 \times 4\frac{3}{4}$ ins., and weighs 16 lbs. The case is of hard maple and conveniently arranged for accessibility of the enclosed parts. The front cover is divided, the upper half being hinged along the upper edge so as to form a flap. Spring hinges keep it closed. The case is divided into two horizontal compartments, the lower half containing a three-bar ringing magneto, induction coil, two dry cells and a 2,500-ohm bell. The upper half contains a hand-telephone set with cord (held in place by spring clips), and also the magneto crank handle and the cord and line plug for making connections as noted at the jacks on the pole. The transmitter and receiver are mounted together on a yoke, so as to be conveniently operated when held in one hand. In the yoke is a push-button (so located that it can be conveniently depressed with the thumb) for closing the battery circuit through the transmitter. When the button is released, this circuit is opened, which prevents the introduction of extraneous noises through the transmitter and economizes the battery consumption.

When not in service this set is carried in a rack at the front end of the car. When a connection with the dispatcher (or other station) is desired, the set is taken from the car and connected to the line by inserting the plug into the jack on one of the poles. The stations are then called by the small hand magneto.

On some electric railway systems where the traffic is very heavy the car or train movements are handled by a dispatcher much as on a steam railway. There is usually this difference, however: on a steam railway the operators who hand the orders to the train crews and set the train order signals are located at regular intervals in the different way stations. On the electric railway there are generally operators only at important points, and therefore it is very desirable that the dispatcher should be able to signal cars at any point and get into communication with the crews of these cars. This can be effected by means of semaphores operated from the dispatcher's office by means of selectors. Any number of these can be placed along the telephone line and the dispatcher can operate any one that he desires without operating any other one.

The semaphore signals may be of the standard types, such as are used in steam railway service, and several of the signal companies make an electrically operated semaphore with the blade moving in either the upper or lower quadrant, as desired. On one such system the semaphore is normally held clear by mechanism operated by battery current. The circuit of the electro-magnet which thus holds the semaphore arm down passes through the back contact of a telegraph relay. The telegraph relay is operated by the telephone selector, which is controlled by the dispatcher.

Now, with this apparatus, to display the "stop" signal at any one station the dispatcher operates the selector at that station. This sends current through the telegraph relay, which breaks the closed circuit of the semaphore mechanism and allows the semaphore arm to go to the "stop" position. The movement of the semaphore arm in turn operates a mechanism which gives the dispatcher a distinctive sound in his head telephone set, informing him that the arm has actually moved to the danger position. This is known as the "answer back" feature. The crew of the next approaching car stop at the signal and get in communication with the dispatcher. They then restore the signal to the "clear" position, or in some systems this may be done by the dispatcher.

It is to be noted that these are in no sense block signals, but simply train-order signals, which are under the direct control of the train dispatcher instead of that of an operator who acts as an intermediary between the dispatcher and the train crew.

Improvements in Underground Conduit Construction for Large Transmission Systems.

With the rapid and continuing growth of the use of electricity for light and power purposes in cities, the construction of the underground conduits to accommodate the large number of cables becomes an increasingly important feature of the distribution system and one which presents a number of special problems. The conduits require not only large capacity but must provide for a high degree of isolation of the cables, so that in case of a burnout in one cable the trouble will not be communicated to other cables and perhaps seriously paralyze an extensive service. This is a specially important problem where the cables carry high-tension currents. The most usual form of construction of the conduits for these transmission systems consists of lines of vitrified duct of rectangular section laid in a trench and surrounded with concrete. Descriptions of the methods of building such conduits were given in our issues of Aug. 20, 1903, and April 14, 1904, and in our issue of July 14, we described a method of concrete duct construction.

There is comparatively little literature upon this subject, and probably few engineers except those engaged directly in electrical work realize the great extent or importance of the conduit systems, or the problems encountered in their construction. One of the primary difficulties is to find room for large conduits in the streets (and especially at street intersections) of large cities, where sewers and various pipe lines and conduits exist. In the rehabilitation of the Chicago street railway system, a special department of the work was the revision of the electric-power distribution system, as noted in an article in our issue of March 10, 1910. This department is in charge of Mr. E. N. Lake, Division Engineer of the Board of Supervising Engineers, and the construction of the conduit systems (with suggestions for improvements in construction designed specially for the protection or isolation of the cables) has been dealt with in a paper read by him before the Western Society of Engineers. This paper, appears in the "Journal" of the Society for June, 1910. It may be noted that the same number of the "Journal" contains a paper on the closely-related subject of "Low-Tension Feeder Systems for Street Railways," by Mr. J. H. Rice, Assistant Division Engineer of the Board of Supervising Engineers. These papers together form a valuable addition to the literature on electrical distribution.

The paper by Mr. Lake is very comprehensive, and we give below some extracts from it, relating particularly to suggested improvements in conduit construction.

The use of ducts for installing electric conductors underground is a practice of comparatively recent years. The ducts were primarily a means for placing cables underground and a convenience for repairing and replacing them. With the large increase in the amounts of energy transmitted, the additional feature—isolation of the cables—is exceedingly important.

The forms at present used are the cement or stone duct, fiber pipe, and the multiple and single form of vitrified clay duct. All of these are installed in the same general manner in concrete, and the methods are practically those used for 15 or more years.

A change or an improvement in conduit construction is sure to encounter strenuous objection all along the line, because of the effect of precedent and inertia. The improvements suggested in this paper are almost all made with reference to the comparatively new ideas of isolation and protection of the cables for which the conduits are built. In this as in all other matters of engineering design, where prevention and insurance are the basis, the precautions taken and the amounts expended for safeguards will depend entirely upon the importance which is attached to these matters by the men who are responsible for the designs and the expenditures. Numerous cases of burnouts which have seriously affected a larger or smaller number of cables adjacent to the defective one can be cited by almost any engineer who has had to deal with underground conduit installations for the distribution of large amounts of electrical energy. So there is apparently a real need of some improvements which will insure against such spread of trouble from one duct to another.

The best heat-resisting and arc-resisting material which is available on a commercial basis at this time is undoubtedly vitrified clay.

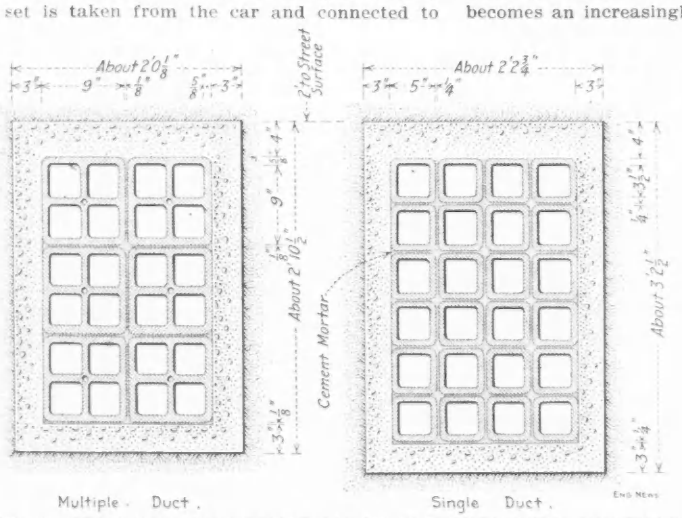


FIG. 1. STANDARD CONSTRUCTION OF CONDUITS WITH MULTIPLE DUCTS AND SINGLE DUCTS.

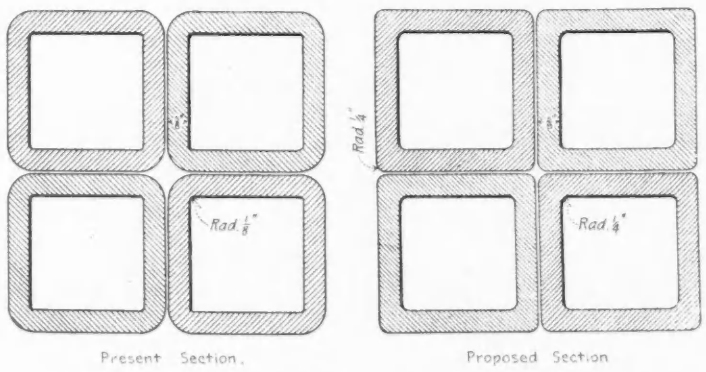


FIG. 2. SECTIONS OF PRESENT AND PROPOSED FORMS OF RECTANGULAR CLAY DUCTS.

When a small quantity of water is closely confined and exposed to sudden and intense heat, it acts with all the violence of an explosive. Due to this so-called explosive power of water, it is believed that the walls of a clay duct (which might otherwise withstand the heat of a burnout) if it contains a sufficient amount of moisture, will be rapidly disintegrated in successive layers by the sudden vaporization of the contained water, under the action of the intense heat of the arc. Hence, that vitrified clay duct is best for large power distribution systems which will absorb the smallest percentage of moisture.

The percentage of absorption permitted will depend upon the importance which the engineer attaches to the matter, and the local conditions as to the kind of duct that can be delivered to the trench at a fair price. One specification used in Chicago requires not more than 2% absorption in 24 hours in the whole piece, and 3% in fragments having about 50% of surfaces glazed. Another specification requires not over 5% absorption, without defining the condition as to the whole or broken pieces.

For telephone cables, and for all cable systems where the number of heavy cables is not large and where no special consideration of the possible spread of trouble from one cable to its neighbors is thought to be necessary, the multiple duct is the logical choice of vitrified clay duct on account of economy of space and convenience in handling. But for systems which carry large groups of cables where tens of thousands of kilowatts may be concentrated upon a burnout, and where cable separation and isolation must be given its due consideration, the single duct construction is believed to be better and is most generally used. These two types of construction are shown in Fig. 1.

The single duct is a much more flexible construction for congested city locations where a great deal of maneuvering is necessary to get around or over, or under, or through obstructions.

The present standard single duct construction, using 3¼ or 3½-in. square bore duct, which is used in most of the larger systems (and shown by Fig. 1), is believed to represent the best engineering practice for large power systems at the present time. This construction, when properly installed, provides two walls of about ⅝-in. each of vitrified clay between adjacent ducts, and for ordinary burnouts this is effective in isolating the trouble. But with very severe burnouts there are certain weaknesses in this form of construction. One of these is due to the form of the corners, which leaves a void of considerable size. Of the space between the corners only the lower half is usually filled with concrete, so that a severe burnout may communicate laterally through the joints in the ducts and longitudinally through the voids, and thus injure cables in adjacent ducts.

One simple improvement in the duct would consist of slightly modifying its shape so as to reduce the void at the corner to such an extent that it may be completely filled by the mortar in the process of laying. By comparing the two designs in Fig. 2 it will be seen that by a slight change in the outside radius of the corner of the duct, the area of the void at the corner of four ducts is very materially reduced. By means of a planimeter, it was found that this corner space was in a ratio of six on the present form to one on the proposed form.

This change in form was submitted to several of the leading duct manufacturers, and while all agreed that it was not impractical to manufacture the duct in the proposed form, there were the usual objections that are urged to any innovation.

To ensure proper isolation of each individual duct in a group of ducts it is essential that the joints should be systematically staggered, but Mr. Lake states that this may not be done properly unless very careful supervision is exercised. In a nine-duct section, the middle duct will have ducts adjacent to its sides and four adjacent to its corners. The first four will usually have the joints staggered with reference to those of the middle duct, but those adjacent to the corners may not be staggered properly. Then a burnout in the middle duct may melt the lead and destroy the insulation of cables in the ducts adjacent to its corners. In conduits having tiers or rows four ducts wide, the joints can be so arranged that those of adjacent ducts will be not less than 4½ ins. apart, or one fourth of the standard length of the ducts. But this is considered too small, while it is extremely difficult to maintain it uniformly under practical conditions of construction. Various arrangements of the ducts with staggered joints are described and illustrated in the paper.

Two plans for securing better isolation are suggested by Mr. Lake. In one of these (A), the single ducts are arranged in groups of four, the ducts being laid in ¼-in. of cement mortar and the groups separated by vertical and horizontal layers of concrete 1 in. thick. In the second plan (B), the vertical layers are dispensed with, the ducts of each horizontal tier being laid with ¼-in. of mortar between them, but the tiers being separated by 1 in. of concrete. This second plan, is shown in Fig. 3, and the following advantages are claimed for it: (1) all joints are staggered at least 9 ins. (or half the length of the duct), (2) the voids at the corners will be largely eliminated (or entirely eliminated with the proposed sharp-corner duct shown in Fig. 2), (3) the cables in each tier are isolated by two ⅝-in. walls of clay and a 1-in. layer of concrete.

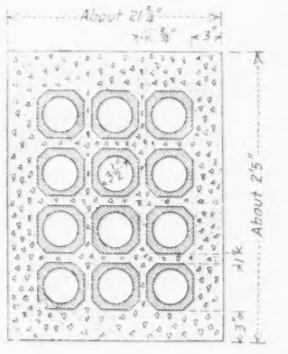


Fig. 4. Section of Conduit With Round-Bore Ducts.

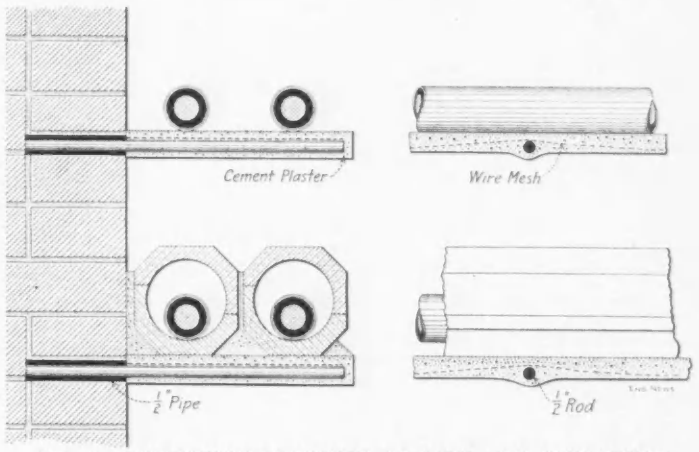


FIG. 5. REINFORCED-CONCRETE SHELVES FOR SUPPORTING CABLES IN MANHOLES.

In regard to the cost, the construction in tiers (as in Fig. 3), involves an average increase of about 6% over the ordinary construction having all the conduits simply laid in cement mortar (Fig. 1). This amounts to about 1 ct. per duct-foot, but Mr. Lake is of opinion that when considered as an additional insurance against cable losses this amount is only a nominal insurance rate. A summary of the prices of the estimates given in the paper is as follows:

No. of ducts in conduit.....	4	12	16	24
	Cost per duct-foot, cts.			
Standard construction	27.6	16.4	14.9	13.3
Group construction (A).....	27.6	17.1	15.6	14.3
Tier construction (B).....	28.1	17.5	16.0	14.5

ROUND-BORE DUCT.—Square-bore duct has come into general use because two or more cables can be installed with greater ease. Round-bore duct of the same size takes less clay and is lighter in weight; there is also less waste due to swelled ends in the drying process, and a smaller number of culls from the kilns, because the shape is such that when piled for burning the heat is distributed more uniformly, and the material is therefore generally better burned and better vitrified.

In ordinary low-tension systems the individual cables seldom carry more than 250 to 500-KW. each. In a high tension system, however, ten to fifteen times this capacity may be transmitted in a single cable. In high-tension systems the cables are usually of such large outside diameter that only one of them can be installed in a duct. For high tension lines and for important low tension lines involving large amounts of power, where a superior construction is desired, a line consisting of round bore single ducts laid in tiers, with an extra space between horizontal ducts, as shown in Fig. 4, would represent, it is believed, the very highest type of construction possible at this date.

A comparative estimate of the cost of this construction and the present standard square duct conduit, assuming identical conditions and taking only 6-duct and 12-duct sections, gives 22 cts. and 16.4 cts. per duct-foot for the square bore, and 23 cts. and 17.6 cts. per duct-foot for the round bore. This small additional cost would often be more than justified on the important lines of a large system.

DUPLEX CONDUIT CONSTRUCTION.—For the safety of the cables on an extensive system it is the practice to have as many routes as possible, so as to minimize the effect of burnouts in interrupting the service. Where practicable, heavy conduit runs are subdivided and built part on one street and part on another so as to make this routing of cables possible. This involves a large additional expense since two 12-duct runs on different streets will cost more for excavation, manholes, concrete, and paving than one 24-duct run on one route.

In order to secure in a large measure the advantages of two separate runs with but very little increase in the

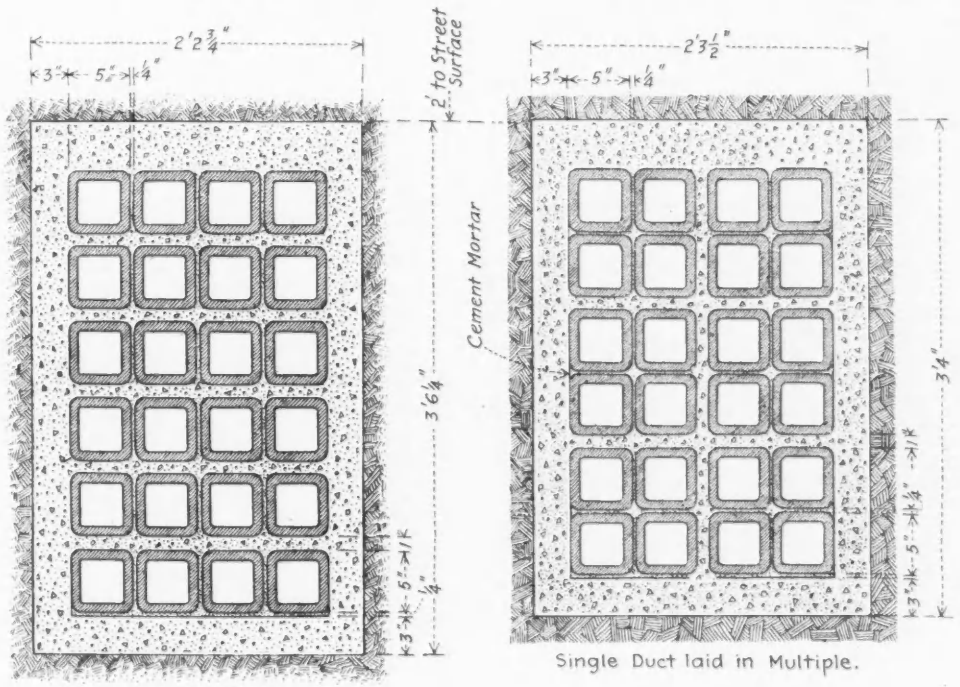


FIG. 3. METHODS OF LAYING SINGLE DUCTS TO ISOLATE THE CABLES IN SEPARATE GROUPS OR TIERS OF DUCTS.

cost, what is styled the duplex construction will be found desirable. This consists simply in dividing the conduit into two sections separated by a substantial vertical barrier of concrete. This may be about 3 ins. thick. Where the ducts enter the manholes this spacing is increased to 6 or 8 ins., and the cables may be trained on opposite walls of the manhole. The additional expense amounts to about ½-ct. per duct-foot on a 24-duct conduit.

LATERALS.—The connections at intervals between the overhead system and the underground system of a street railway are made by means of taps or laterals extending from a manhole through a suitable duct up to the overhead conductor on the pole. An apparent cause of failure is electrolysis, the effects of which are often found in the bends of the iron pipe through which the lateral connections are made. For the vertical portion upon the pole there appears to be nothing better than iron pipe, but the run from the manhole to the pole may be of 3-in. sewer pipe or 3½-in. standard duct.

FIREPROOF SHELVES IN MANHOLES.—Sheet steel has been used for shelves in manholes, but it has been found that if the metal is thick enough to give a reasonable degree of rigidity, the cutting and fitting is difficult and expensive. In case of a severe burnout the steel, because of its good conducting qualities and relatively low melting point, tends to spread and augment the trouble rather than localize it.

The upper part of Fig. 5 shows a shelf construction that is less expensive, more flexible, and with the metal parts entirely covered with cement, so that it is a combined shelf and barrier. The wire mesh may be had in rolls and can be easily cut and fitted, and any man familiar with the use of the trowel can (with a little plasterer's hair mixed into the cement mortar) readily lay these shelves in place. Cables are thus effectually separated into groups by a substantial horizontal barrier of fireproof material.

Where only a small number of relatively unimportant cables are involved this construction gives a degree of isolation that is quite sufficient. For a larger group of more important cables, the split duct construction shown in the lower part of Fig. 5, in addition to the reinforced-cement shelves, can be used advantageously. Split duct is made by scoring the walls of the duct as it is excluded from the presses, so that after being burned it can be shipped to the work in whole pieces and there broken into parts for use.

Discussion.

MR. D. W. ROPER.*—I disagree with the statement that vitrified clay is the best non-conductor of heat and the best arc-resisting material. Recently I endeavored to find some scientific data for comparing concrete with tile in specific heat, in heat conductivity, and in resistance to high temperatures. The experiments forming the basis of the several articles indicated that concrete construction was a poorer heat-conducting material than vitrified tile. The Commonwealth-Edison Co. has had an experience in a burnout that illustrates this.

Mr. Lake spoke of the desirability of getting vitrified clay which is free from moisture, because of the difficulties which would be caused by an arc or other sources of heat in the duct or in the conduit construction. This freedom from moisture is also desirable from the point of view of the freezing of the ground. We had a case where the ducts were laid, as most of our conduits are, within freezing depth (that is, 30 ins., more or less) of the surface of the street. These ducts had a high percentage of absorption. Some years later we found that this conduit was so badly broken that it could not be handled; there were a series of fractures running all through the tile like crackle ware. It apparently had been ruptured by freezing, and although it had not so broken that it collapsed and filled up the hole, it showed that the structure was considerably weakened and might have caused trouble later.

As to the ducts laid in horizontal rows separated by an inch of concrete (Fig. 3), it is very difficult to secure uniform thickness in spreading a single layer of concrete, and to keep the ducts in alinement after they are laid. After the trench is excavated, 3 ins. of concrete is laid on the bottom, then a layer of ducts, then 1 in. of concrete and another layer of ducts. But the workmen will walk on this latter layer of tile, resting on an inch of concrete which has not set, with apparently nothing to hold the ducts in alinement except gravity and the friction on the concrete.

The result is that with that kind of construction, instead of getting the expected 3½-in. opening, it is found impossible sometimes to pull through a 3-in. mandrel and difficult to get a 2½-in. mandrel through. The question of alinement is one of the hardest problems in the single-duct tile construction, and our experience is that we have less difficulty drawing our cables into 3¼-in. round-section concrete pipe, which is in more accurate alinement, than we have in 3½-in. square-bore vitrified tile.

*Assistant to the Operating Engineer, Commonwealth Edison Co., Chicago.

Occasionally we have had difficulty in drawing cables into the tile ducts, and the foremen have brought sections of such cables into the office which looked as if they had a square section; that is, the ducts would be offset in one place so that the cable would be squeezed and flattened, and in another place the cable would twist or the alinement would be offset in the other direction, resulting in a section that looks like a square cable with rounded corners; not 3½ ins. square, but 2½ ins. or thereabouts.

The statement was also made that the square-bore section offers less resistance when two or more cables are being drawn in. That is the reverse of our experience.

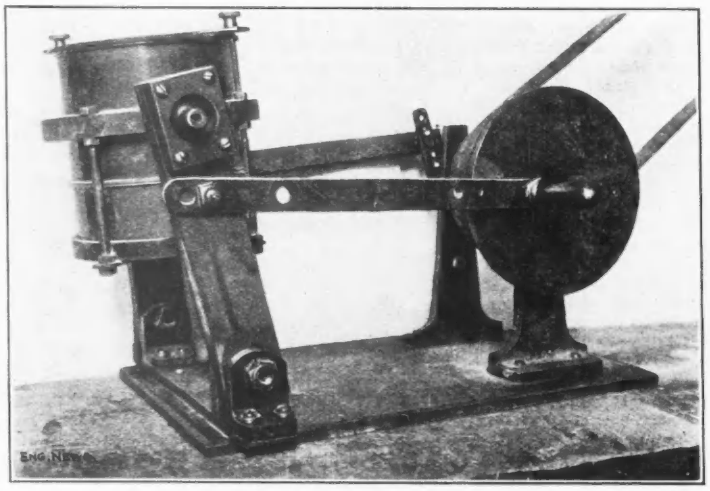


FIG. 1. VIEW OF EXPERIMENTAL CEMENT SHAKER SIEVE IN USE AT LAFAYETTE COLLEGE.

The men on the ground who have actual charge of installing these cables say that the large cables (2 to 2¾ ins.) are drawn into the square ducts with less friction than into the round ducts, the arc of contact being less when the cable rests on a flat surface than on a cylindrical surface. This refers to conduits in which there is no difficulty with the offsets noted above.

MR. F. W. DARLING.*—In regard to the voids in connection with the square duct. After Mr. Lake sent us that letter asking for information relative to manufacturing the square tile with a very small radius on the outside corners, we attempted to make some, using an old die and filing out the corners. We found that such tile could be easily made, but that if the clay is run stiff and thick, requiring but a short time in the dry room and thus reducing the cost, it roughs up or scales on the corners. That does not injure the tile, however; the outside is simply roughened up, with a little peeling and scaling. The inside remains perfectly smooth. Of course, the inspectors might infer that those cracks went through.

A well vitrified clay conduit, in its manufacture, is burned to a temperature of approximately 3,000°. It is supposed to stand a temperature of about 4,000° before giving way. Under usual and ordinary circumstances, cement products cannot stand a temperature of over 1,800° without having the lime salts broken up and dissipated, reducing the cement mass to a condition about as friable as soft plaster. With this in mind, it seems hardly fair to accept the single instance which Mr. Roper mentioned as a proof of the test.

*Clay Products Co., Chicago.

A Machine-Driven Cement Sieve.*

At the last annual meeting of the American Society for Testing Materials, Prof. J. M. Porter, of Lafayette College, described a number of the different devices which had been installed under his direction in the testing laboratories of his college. Among these was the mechanical cement shaker-sieve to be used in the testing of cement which is described below.

Although the standard specifications most generally in use in this country recommend that hand-sieving only be used to determine the fineness of cement, a number of laboratories have built shaker-sieves which can be connected up to power and thus save much manual labor in this step of the testing process as well as ensure a more uniform test for fineness. The machine devised by Prof. Porter is shown in the view in Fig. 1 and in the original design in Fig. 2. Some few changes were made in building the machine shown in the half-tone, so it does not entirely agree with the drawings. It consists of two rocker arms supporting a ring frame which surrounds and supports the sieves and receiving pan. One rocker arm (the far one in the figures) is attached to the ring by a double hinge in such a manner that the ring may have a rotary motion in the plane of the arm and also at right angles to the same, but no longitudinal move-

*See description of another type of mechanical cement sieve by Prof. T. R. Lawson in Engineering News, Dec. 30, 1909, p. 728.

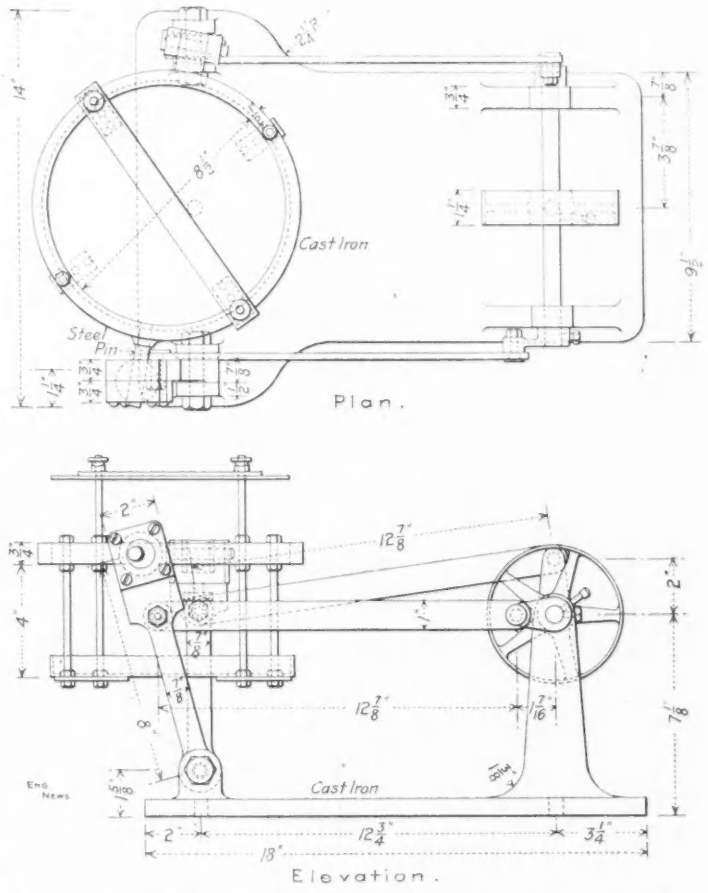


FIG. 2. DETAILS OF LAFAYETTE COLLEGE CEMENT SIEVE FOR TESTING CEMENT AS ORIGINALLY DESIGNED.

ment. The other rocker supports the opposite side of the ring by a short shaft projecting through a ball which forms part of a ball and socket bearing, thus allowing rotary as well as longitudinal motion. The two rocker arms are driven by means of short connecting rods attached to cranks having different throws, the near one being 1½ ins. and the far one 2 ins. The far crank is set with about 80° angular advance.

The apparatus is driven by belt as shown in Fig. 1, at 170 r. p. m. and accomplishes results in about one-third of the time required by hand.

The apparatus shown is only an experimental one and it is hoped that some improvement can be made in the next one that is built. Prof. Porter states that he has in mind changes as follows: to rotate the sieves about their own vertical axis, to shorten the connecting rods and add another set of rocker arms and sieves to the right of the shaft, driving them from the same cranks.

An Electric Freight Locomotive for City Lines.*

A new design of electric locomotive has been made by the General Electric and American Locomotive companies, of Schenectady, N. Y., for the Transit Development Co., of Brooklyn, New York City, to be used for switching and freight service over the lines of the Brooklyn Rapid Transit Co. The Transit Development Co. was organized to hold title to real estate and other property needed for the street-railway service, to sell power and to do the maintenance and repair work of all the systems of the Brooklyn Rapid Transit Co.

It is reported that the increasing and peculiar demands for freight service over the Brooklyn lines has made necessary a switching locomotive with special features. The collection of heavy freight from terminals and docks, and the occasional short sharp grades encountered along the water front, have necessitated a heavy type, while the distribution of freight to shops and factories as far distant as South Brooklyn and Coney Island, and the requirement that such service shall not interfere with passenger service have made it essential that the locomotive should be worked at high speed. To fulfill all these requirements it was necessary that higher power motors be installed than would be used for ordinary switching service.

The general appearance of the new design is that of the familiar electric switching locomotive with center cab and sloping end compartments. The trucks are similar to the standard two-bar equalizer type built for the Brooklyn elevated-road motor cars; all the wearing parts are interchangeable except springs, equalizers and parts necessarily heavier to bear the increased loads.

The locomotive platform is built up of longitudinal sills of 10-in. channels extending the whole length and is fitted with cast-iron end frames and heavy built-up bolsters, of 17 ¾ × 1-in. plates with center-pin castings riveted to them. Drawhead castings are attached to the center sills and end frames for greater rigidity. The draft rigging consists of M. C. B. couplers with 5 × 5-in. shanks. The whole platform is squared and braced by heavy floor plates extending the whole width of the locomotive and riveted to the longitudinal sills. The locomotive has been designed for a weight of 55 tons and, as ballast has been required to bring it up to this weight, 10 × 2-in. bars run the whole length of the platform, notched over the bolster plates, and bolted to the longitudinal sills. The main operating cab is in the center of the platform with auxiliary end compartments and side platforms extending out toward the ends of the locomotive. This type of cab gives an extended view to the operator in all directions from his operating seat and on this account has been used instead of the box type used on previous Brooklyn Rapid Transit locomotives. The auxiliary compartments contain sand boxes, air reservoirs, rheostats and contactors.

There are four commutating-pole motors, each rated to develop 225 HP. for one hour without overheating. A tractive effort of 27,500 lbs. can be secured at 7 mi. per hr. or 16,880 lbs. at 20 mi. per hr. or 6,000 lbs. at 30 mi. per hr. This ensures a 500-ton train being pulled up a 2% grade or hauled on a level at 30 mi. per hr., maximum speed.

The control is a distinctive feature. A "multiple-unit" system has been furnished providing three motor groupings with a total of 18 steps, seven with all four motors in series, six for series-parallel and five with all motors in parallel. Such a three-speed control is demanded by the conditions of service when a high speed is required for level running with through freight and an economical low speed is needed for heavy switching work. Three-speed control has been used previously on locomotives for heavy railway service, but has not been generally required upon electric roads. The type of control required for electric-locomotive service differs from that for ordinary car service, with varying load on the driving wheels, in that it must be arranged to work the motors as close as safely possible to the slipping point of the wheels throughout the entire range of acceleration without sudden fluctuations of tractive effort. In the present case it is stated that the large number of steps and the rheostat divisions have been very carefully designed and proportioned to the weight of the locomotive. Also, the transfer from one combination of motors to another has been arranged in connection with the rheostat steps so as to secure a uniform gradation of tractive effort throughout the entire range of control.

Controllers are placed at the engine-runner's seats in diagonally opposite corners of the cab. Third rail shoes are provided for use on the elevated lines; there are two complete trolley poles designed to be used simultaneously against the same overhead wire to limit the current at each point of contact with the overhead circuit.

The following table gives the principal dimensions of the locomotive:

Number of motors	4
Gear ratio	3.37
Number of driving wheels	8
Diameter	34 ins.
Total wheel base	22 ft. 2 ins.
Rigid wheel base	6 ft. 8 ins.
Length inside knuckles	31 ft. 1 in.
Length main cab	8 ft.
Length of cab over all	27 ft.
Total weight	110,000 lbs.
Traction effort at 25% co-eff.	27,500 lbs.
Speed at max. tractive effort	7 m. per hr.

*From information supplied by the General Electric Co., Schenectady, N. Y.

A 12-IN. RIFLE BURST at the forward turret of the U. S. Battleship "Georgia" Sept. 23 during target practice off the Virginia Capes. The gun burst at a point near the muzzle, outside the turret, and no one was injured.

SPREADING RAILS are designated in press reports as the cause of a wreck on the Chicago & Erie R. R. Sept. 21 about nine miles west of Lima, Ohio. One passenger was killed and 25 injured. The train was eastbound passenger No. 4. The accident took place on a curve where the tracks had been recently raised a few inches. The baggage car and four day coaches were overturned, while two Pullman cars at the rear of the train stayed on the rails.

A COLLISION IN AUSTRIA Sept. 20 wrecked two express trains near Rottenmann. Eleven persons were killed and 25 others injured.

A RAILWAY WRECK NEAR CLAYTON, KAN., Sept. 23, on the Chicago, Rock Island & Pacific Ry., caused the deaths of 16 persons and the injury of a dozen others. The wrecked train was No. 27 from Kansas City, due in Denver at 8.25 a. m. Reports agree in ascribing the accident to a washout caused by a cloudburst, but there is some divergence as to details. According to one account, a 60-ft. trestle was carried away, while another has it that a thousand feet of track was washed out at a fill.

A BUTTING COLLISION OF TROLLEY CARS near Kingsland, Ind., Sept. 21, on the Fort Wayne, Bluffton & Marion Ry. (operated by the Fort Wayne & Wabash Valley Traction Co.) killed outright 40 persons and seriously injured nine or ten others, one of whom has since died. The killed and injured were all in one car, a local, bound from Bluffton for Fort Wayne. The other car was a limited carrying no passengers. The collision occurred on a sharp curve, on which the view ahead was cut off by a grove of trees at the concave side. Thus the motormen had very little warning and the cars met practically at full speed, so that they were telescoped together for about half their length. To this and to the fact that the local car was fully loaded, carrying about 60 passengers, may be ascribed the unusually large list of fatalities. The accident is ascribed by railway officials to disregard of meeting orders on the part of the motorman of the limited car. Both motormen jumped before the crash and escaped with severe injuries.

A very similar accident Sept. 24 on the line of the Indiana Union Traction Co., two miles north of Tipton, Ind., killed six persons and injured twelve others. A south-bound freight met a north-bound limited passenger car on a curve. As the Kingsland accident, the view ahead at the curve was obscured by a clump of trees. The accident is ascribed to overrunning of meeting orders by the freight. Both motormen escaped by jumping.

A FLIGHT OVER THE ALPS, at Simplon Pass, was made by Geo. Chavez on Sept. 23, with a monoplane flyer. He intended to fly from Brig to Milan but in descending at Domodossola, Italy, the machine collapsed 30 ft. above ground, crushing him under the engine. He was so badly injured that he died on Sept. 27.

An aviator, named Poillot, was killed at Carlas, France, on Sept. 25, while making a flight with one passenger in a monoplane. It is reported that the cloth covering of the main planes ripped.

CHOLERA IN RUSSIA is showing a decided decline of late. A press despatch from St. Petersburg, dated Sept. 23, states that available figures showed a total of 191,076 cases, with 88,716 deaths in Russia, thus far this year. New areas were being invaded. It may be added that press despatches from other parts of Europe show that the disease has been gaining some foothold outside of Russia, chiefly in Italy, it would appear. Germany, France and England, and, in fact, all the countries of northwestern Europe seem to have been kept practically free from the disease.

THE SERVICES OF A NAVAL ARCHITECT are wanted by the city of Portland, Ore., to prepare plans, specifications and estimates, and to superintend the construction of a steel fire boat "to cost approximately $125,000." "Sealed proposals" will be received by "A. L. Barbur, City Auditor of Portland, Ore.," until Oct. 13, 1910.

WHAT IS PHILADELPHIA DOING? This very pertinent question for Philadelphia or any other city is to be answered in the case of Philadelphia by a series of addresses to be delivered at 14 Saturday luncheons and one evening meeting of the City Club of Philadelphia during the coming months. The water and milk supplies, health, streets, park, lighting, educational, and other departments of the city will all be passed in review, as will also transportation facilities by land and water. The closing subject on the list is city plan and development which has been made broad enough to include consideration of almost all the agencies affecting the general plan and future of the city. The outline of the various topics named shows that each one of them will be considered under the general sub-headings of Functions, Public Agencies, and Private Agencies. Each one of these sub-topics has been further subdivided. It may thus be seen that by careful formulation of the method of treatment of each subject to be considered, the season's program will have a unity of treatment which could not otherwise be expected.

AMENDED SEWAGE DISPOSAL PLANS of the City of Rochester, N. Y., have been approved by the State Department of Health. The various engineering reports connected with the return of these plans to the city authorities of Rochester for amendment were abstracted at length in our issue of Aug. 11, 1910, p. 157. The principal change demanded by the State Department of Health was an increase in the capacity of the settling basins, or, more properly, the grit chambers. Apparently, in the belief that this capacity could not be increased without destroying the real purpose of the grit chambers, the consulting engineer, Mr. Emil Kuichling, M. Am. Soc. C. E., of New York City, added a second set of settling basins which in their entirety have doubled the settling capacity of the original grit chambers. In other words, the total settling capacity will be three times as great as was originally proposed, but by making the additional basins independent of the grit chambers the function of the latter will not be destroyed by so increasing their capacity as to detain considerable portions of organic matter. For the screens of 1/10-in. nominal mesh, proposed originally, others of ⅛-in. mesh will be substituted. The net result of the changes in the plans which, we understand, include some minor

features not mentioned above, will be $150,000, but on adding the capitalized valuation of the increased operating expenses the result will net the equivalent of a total of $600,000 additional first cost.

INFORMATION REGARDING THE WATER PURIFICation plant of the Indianapolis Water Co. was circulated on small blue prints at the recent convention of the Central States Water Works Association at Indianapolis, Ind. The plant includes coagulation, sedimentation and slow sand filtration in the order named. Water is taken from White River and Fall Creek. A chemical house is arranged to supply either alum or iron and lime up to a water volume of 30,000,000 gals. per day. The settling capacity is 45,000,000 gals., allowing from 1½ to 2 days' sedimentation after coagulation. There are six cover filter beds, each .776-acre in area, having a total capacity of 21,000,000 gals. per day. During the year 1909 a total of 5,708,000,000 gals. of water were filtered at an average rate of 3,660,000 acres per day while, during the first five months of 1910, the rate averaged 4,240,000 gals. The last-named rate, the company states, was shown to be higher "than any other slow sand filter in the country." The bacterial results for 1909 were as follows:

	Raw water	Settled water.	Filtered water.
Bacteria per c.c.....	4,255	1,255	54
Turbidity	59	16	0

PROPERTY RIGHT IN MACHINERY was the subject of a court decision in Germany recently, as noted in the September, 1910, issue of "Technik und Wirtschaft." Shafting and other transmission plant was involved. This equipment had been supplied to a sawmill operated by water-power, but apparently had not yet been paid for when the property was put under foreclosure. The seller of the transmission equipment sued to regain possession, having retained title to the goods. The foreclosing creditor claimed that the equipment was an integral part of the mill and therefore could not be reclaimed. The court decided against him, however. The decision set forth that the building and water-power form an indivisible unit, whereas the shafting, etc., like the machines, can at any time be dismounted readily and taken away without injuring the building. The shafting and auxiliaries being stock articles, used in commercial sizes, it could not be held that by their installation they had become so specially adapted to this particular building as to become incapable of separate use. The transmission equipment therefore was not robbed of its status as movable property by being installed in the mill.

A GIRDER ROOF FOR CARRYING A 12-STORY BUILDING.—Just east of the new Pennsylvania R. R. Terminal station in New York City, the approach tunnels in 32d and 33d Sts. pass under private property. In order to leave this property useful for building purposes, the tunnels at these points were provided with a roof capable of carrying a load of 10,000 lbs. per sq. ft., the equivalent of a 12-story building. Plate-girders 9 ft. deep spaced as close as 24 ins. c. to c. form this roof. As the tunnel at this point is on curve and its walls are not parallel, the span and skew of the girders varies. The maximum span is 77 ft. They were encased in concrete as a preservative.

Personals.

Mr. Kitchell M. Boorman has opened an office as a consulting civil engineer at 11 Cliff St., New York City.

Mr. H. B. McMaster has resigned his position as Assistant Manager of Sales of the Youngstown Sheet & Tube Co., Youngstown, Ohio.

Mr. Henry B. Seaman, M. Am. Soc. C. E., has resigned his position as Chief Engineer of the New York Public Service Commission, First District.

Mr. George S. Rice, M. Am. Soc. C. E., has resigned his position as Engineer of Subway Construction, New York Public Service Commission, First District, and will enter private practice.

Mr. George C. D. Lenth has been appointed Chief Engineer of the Board of Local Improvements, Chicago. He succeeds Mr. C. D. Hill, who has been appointed Superintendent of Sewers.

Mr. M. A. Kinney, Master Mechanic of the Hocking Valley Ry. at Columbus, Ohio, has been promoted to be Superintendent of Motive Power to succeed Mr. G. J. De Vilbiss, who was recently killed in the derailment noted in our issue of Sept. 22.

Mr. Edward B. Guthrie, M. Am. Soc. C. E., and Mr. George C. Diehl have dissolved their partnership of Guthrie & Diehl, which has conducted a civil engineering practice in Buffalo, N. Y., for the past twelve years. Each of the former partners will open a separate office, Mr. Guthrie at 445 Ellicott Square and Mr. Diehl at 575 Ellicott Square, Buffalo, N. Y.

Mr. Lucius Tuttle, who recently resigned his position as President of the Boston & Maine R. R., has been voted an additional year's salary ($50,000) by the directors of that road in appreciation of his services. He has been asked to remain in an advisory capacity at a salary of $10,000 per year.

George A. Johnson, Assoc. M. Am. Soc. C. E., until recently a member of the firm of Hering & Fuller, New York City, and William B. Fuller, M. Am. Soc. C. E., formerly Engineer-in-Charge of the Filtration Division, Department of Water Supply, Gas and Electricity of New York City, have formed a partnership under the firm name of Johnson & Fuller, with temporary offices at 150 Nassau St., New York City.

Mr. Johnson has been a member of the firm of Hering & Fuller since May, 1908, and has been associated with the firm much longer. He served under Mr. Geo. W. Fuller, M. Am. Soc. C. E., in both the Louisville and the Cincinnati water purification experiments, from 1895 to 1898, and was associated with Mr. Geo. C. Whipple, M. Am. Soc. C. E., in a sanitary survey of the Hudson River drainage area for the Burr-Hering-Freeman Commission on Additional Water Supply for New York City. In 1904, Mr. Johnson became Engineer-in-Charge of the sewage and water purification investigations at Columbus, Ohio.

Mr. Wm. B. Fuller was resident engineer of the Albany slow sand filtration plant under Mr. Allen Hazen, M. Am. Soc. C. E., and occupied a like position under Mr. Geo. W. Fuller on the mechanical filtration plant built at Little Falls, N. J., for the East Jersey Water Co. to treat the water-supply of Paterson and numerous other communities. He was Department Engineer on Filtration under the Burr-Hering-Freeman Commission, already mentioned. In these and other connections he has specialized in concrete design and construction.

Obituary.

John Franklin Robinson, of Pittsburg, Pa., died Sept. 9. He was the senior member of the firm of Robinson & Orr, dealers in second-hand engineering materials.

William Harrison Foss, an assistant engineer in the Department of Streets of Boston, Mass., died Sept. 21 at his home in Brighton, Mass., aged 73 years. He served in the U. S. Engineer Corps during the Civil War and had been in the employ of the city of Boston continuously, with the exception of two or three years, since 1875.

William Harmon Niles, formerly Professor of Geology at the Massachusetts Institute of Technology, died in Boston, Sept. 13. He was born at Northampton, Mass., in 1838 and was a graduate of the Lawrence Scientific School in the class of 1866. He was at the head of the department of geology at the Massachusetts Institute from 1878-1902.

Clay Belsley, City Engineer of Peoria, Ill., died in that city on Sept. 3. He was born at Spring Bay, Ill., on Jan. 28, 1873. He was a graduate of the Peoria High School, and also graduated in Mechanical Engineering at Cornell University in 1898. Shortly after leaving Cornell he went into consulting engineering work, and opened an office for that purpose in Peoria. He was appointed City Engineer in May, 1909.

Gilbert W. Sanborn, an inspector on contract No. 15 of the Catskill Aqueduct near Gardiner, N. Y., was instantly killed Sept. 22 by a knife thrust. The wound which caused his death was received by accident while he was endeavoring to protect one of the contractor's men from the attacks of a fellow workmen. Mr. Sanborn was a brother of Mr. James F. Sanborn, Assoc. M. Am. Soc. C. E., Division Engineer of the Board of Water Supply of New York City. The interment was at the former home of the deceased in Templeton, Mass., Sept. 24.

Samuel M. Rowe, M. Am. Soc. C. E., Consulting Engineer, Chicago, died May 21, aged nearly 80 years. From 1867 to 1876 he was engaged on construction of the Chicago, Burlington & Quincy R. R., and from 1878 to 1891 he was engaged on the Atchison, Topeka & Santa Fe Ry. For four years of the latter period he was Chief Engineer of the Atlantic & Pacific Ry., and the Red Rock cantilever bridge at The Needles was built under his direction. From 1891 to 1896 he was engaged on topographical survey work in New Mexico, on the Lake St. and Northwestern elevated railways of Chicago, and on the Halsted St. lift bridge, Chicago. He was largely instrumental in the establishing of a cement testing laboratory by the city of Chicago, and was at one time connected with the Forest Bureau of the U. S. Department of Agriculture. He wrote a number of articles and papers on timber treatment, and also a "Hand-Book of Timber Preservation." He became a member of the American Society of Civil Engineers in 1885, and in 1892 a paper by him (and by S. W. Robinson and H. H. Quimby) on "The Red Rock Cantilever Bridge" was awarded the society's Rowland prize. He was a member also of the Western Society of Engineers, the Wood Preservers' Association, and the American Railway Engineering and Maintenance-of-Way Association. In spite of his 80 years, Mr. Rowe was engaged in active work to within a short time of his death. He had been in private practice since 1896, and had made a specialty of timber preservation and the designs of plants for preservative treatment. During this time he acted in a consulting capacity for a number of railways.

Engineering Societies.

COMING MEETINGS.

AMERICAN STREET AND INTERURBAN RAILWAY ASSOCIATION.
Oct. 10-14. Annual convention at Atlantic City, N. J. Secy., H. C. Donecker, 29 West 39th St., New York City.

AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS.
Oct. 11-14. Annual convention at Erie, Pa. Secy., A. P. Folwell, 239 West 39th St., New York City.

RAILWAY SIGNAL ASSOCIATION.
Oct. 11-13. Annual meeting at Richmond, Va. Secy., C. C. Rosenberg, Bethlehem, Pa.

AMERICAN ELECTROCHEMICAL SOCIETY.
Oct. 13-15. Annual meeting at Chicago, Ill. Secy., Jos. W. Richards, Lehigh University, South Bethlehem, Pa.

AMERICAN RAILWAY BRIDGE AND BUILDING ASSOCIATION.
Oct. 18-20. Annual convention at Denver, Colo. Secy., C. A. Lichty, Chicago & Northwestern Ry., Chicago, Ill.

AMERICAN GAS INSTITUTE.
Oct. 19-21. Annual meeting at New York City. Secy., A. B. Beadle, 29 West 39th St., New York City.

AMERICAN INSTITUTE OF MINING ENGINEERS.
Oct. 21-Nov. 15. Canal Zone meeting. Secy., R. W. Raymond, 29 West 39th St., New York City.

ILLUMINATING ENGINEERING SOCIETY.
Oct. 24. Annual convention at Baltimore, Md. Secy., P. S. Millar, 29 West 39th St., New York City.

NATIONAL ASSOCIATION OF RAILWAY COMMISSIONERS.
Nov. 15. Annual meeting at Washington, D. C. Secy., Wm. H. Connelly, Interstate Commerce Commission, Washington, D. C.

AMERICAN RAILWAY ASSOCIATION.
Nov. 16. Semi-annual meeting at St. Louis, Mo. Secy., W. F. Allen, 24 Park Pl., New York City.

SOCIETY OF NAVAL ARCHITECTS AND MARINE ENGINEERS.
Nov. 17-18. Annual meeting at New York City. Secy., W. J. Baxter, 29 West 39th St., New York City.

NATIONAL SOCIETY FOR THE PROMOTION OF INDUSTRIAL EDUCATION.
Nov. 17-19. Annual convention at Boston, Mass. Secy., James C. Monaghan, 20 West 44th St., New York City.

NEW JERSEY SANITARY ASSOCIATION.
Dec. 2-3. Annual meeting at Lakewood, N. J. Secy., J. A. Exton, 75 Beech St., Arlington, N. J.

AMERICAN SOCIETY OF REFRIGERATING ENGINEERS.
Dec. 5-6. Annual meeting at New York City. Secy., W. H. Ross, 154 Nassau St., New York City.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.
Dec. 6-9. Annual meeting at New York City. Secy., C. W. Rice, 29 West 39th St., New York City.

AMERICAN INSTITUTE OF CHEMICAL ENGINEERS.
Dec. 7-9. Annual meeting at New York City. Secy., J. C. Olsen, Polytechnic Institute, Brooklyn, N. Y.

LAKES-TO-THE-GULF DEEP WATERWAY ASSOCIATION.—The fifth annual convention will be held at St. Louis, Mo., Nov. 25-26. The secretary is Mr. Thos. H. Lovelace, 909 New Bank of Commerce Bldg., St. Louis, Mo.

ENGINEERS' SOCIETY OF WESTERN PENNSYLVANIA.—At the regular monthly meeting of Sept. 20, papers were presented on "Contractors' Plants," by Frederick E. Field, M. Am. Soc. C. E., and "Forms for Concrete," by J. D. Stevenson.

CHICAGO ARCHITECTURAL SKETCH CLUB.—A reunion of the members of this club, which was organized at the time of the Chicago World's Fair, is to be held in February, 1911. The secretary is Mr. C. H. B. Schaefer, 2919 Flournoy St., Chicago, Ill.

AMERICAN RAILWAY BRIDGE AND BUILDING ASSOCIATION.—Subjects for report and discussion at the 20th annual convention at Denver, Colo., Oct. 18-20, are announced as follows: (The name given after each subject is that of the chairman of the committee to which the subject is assigned.) 1. Protection of embankments against currents and methods of restoring same when washed out, E. L. Loftus, Vicksburg, Miss.; (2) How to prevent iron pipe culverts from pulling apart in soft ground, and how best to repair them when pulled apart, A. A. Page, Wilmington, Mass.; (3) Reinforcement and waterproofing of concrete in railway construction, C. W. Richey, Pittsburg, Pa.; (4) Arrangement of railway buildings and platforms for small towns, C. H. Fake, Bonne Terre, Mo.; (5) Best method of determining proper dimensions of openings for waterways, W. T. Main, Chicago, Ill.; (6) Best method of obtaining elevation on curves on bridges and trestles, J. P. Snow, Boston, Mass.; (7) Best method of numbering bridges, I. F. Stern, Chicago, Ill.; (8) The economy and practicability of wire glass in roundhouses, shops and station buildings, E. E. Wilson, New York; (9) The best style and dimensions of hoops for water tanks of 50,000 to 100,000 gals. capacity, F. E. Weise, Chicago, Ill.; (10) Plans for fireproof oil houses for storing large quantities of oil at principal terminal stations, G. W. Rear, San Francisco, Cal.